Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
Proposed SR 823 Over SR 140 (Webster Street)
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, Ohio 43017

Ohio Department of Transportation
District 9

Prepared by:

DLZ

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DLZ Job No. 0121-3070.03

January 11, 2007

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

PROPOSED SR 823 OVER SR 140 (WEBSTER STREET)

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

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1.0 INTRODUCTION

This report includes the findings of evaluation of foundations and mechanically stabilized earth (MSE) retaining walls for the structure at the above-referenced location of the project. The findings included in this report pertain to the structure at the intersection of the proposed SR 823 and SR 140 (Webster St.) only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing two structures, northbound and southbound structures, respectively for proposed SR 823 over SR 140 (Webster St.). The two structures as planned, are single-span structures using MSE walls to hold back the roadway embankments and contain the abutments.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the structure foundations, MSE walls, and the approach embankments. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, considered the findings, and prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 GENERAL PROJECT INFORMATION

It is understood that the plan location of the bridge structure for the proposed SR 823 over SR 140 (Webster St.) has not changed from the approved location, as shown on the Plan and Profile drawing in Appendix I. It is understood that MSE walls will be placed at approximately stations 61+70.5 and 62+62.5 to contain the abutments and hold back the roadway embankment for proposed SR 823. Furthermore, it is understood that spread footings will be used to support the abutments of the proposed structures.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the embankment at stations 61+62.2 (Rear Abutment) and 62+70.2 (Forward Abutment) will be approximately 39.5 and 32.0 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of SR 823 and the approximate existing grade along the proposed alignment.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concept are changed or differ from that assumed, DLZ should be informed of the changes so that recommendations and conclusions presented in this report may be revised as necessary.

3.0 FIELD EXPLORATION

The field exploration consisted in part of three final and three preliminary structural borings. Borings B-15 through B-17 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over SR 140 (Webster St.), as shown on the structural site plan in Appendix I. Final structure borings, B-15 through B-17 were drilled on September 19 and 20, 2006. Structure borings (TR-43 through TR-45) were drilled for a previous design configuration. These borings were drilled between February 2 and 24, 2005. Boring logs for borings TR-43 through TR-45, and B-15 through B-17 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were planned and staked in the field by representatives of DLZ. The surveyed locations and ground surface elevations of the borings were determined by representatives from Lockwood, Lanier, Mathias & Noland, Inc. (2LMN).

4.0 FINDINGS

4.1 Geology of the Site

The area of this structure is characterized by gently sloping to steeply sloping topography. The project area is located in the Shawnee-Mississippián Plateau of the unglaciated portion of the Appalachian Plateau Physiographic Region. The Shawnee-Mississippian Plateau is characterized by Devonian aged to Pennsylvanian aged rocks and contains residual colluvial, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Soils at the rear abutment location are composed primarily of residual and colluvial soils. These soils are generally thin, covering moderate to steep slopes. At the forward abutment, residual and lacustrine soils were encountered. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. Bedrock within the structure area is primarily sandstone of the Logan Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes roughly above elevation 770 in this area.

4.2 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix II. Laboratory test results are presented on the boring logs and also in Appendix III.

4.2.1 Soil Conditions

In general, the subsoil stratigraphy consisted of shallow surficial materials consisting of topsoil or asphalt concrete pavement underlain by native cohesive and granular soil deposits and sandstone.

Boring B-15 was drilled for the rear abutment of the approved (final) structure configuration. Similarly, borings B-16 and B-17 were drilled for the forward abutment of the approved (final) structure configuration. Borings TR-44 and TR-45 were considered in the evaluation of the rear abutment location, while boring TR-43 was considered in the evaluation of the forward abutment location.

Borings TR-45 and B-15 encountered surficial material consisting of 2 inches of topsoil while boring TR-44 encountered 12 inches of asphalt concrete pavement. The topsoil in boring TR-45 was underlain by bedrock. Borings TR-43, TR-44, and B-15 through B-17 encountered native cohesive and granular soil deposits below the surficial material or the ground surface. The cohesive deposits consisted mainly of stiff to hard silt (A-4b), very stiff silt and clay (A-6a), stiff to very stiff silty clay (A-6b), and very stiff clay (A-7-6), while the granular soil deposits consisted mainly of loose to medium dense sandy silt (A-4a). The native soil deposits extended to depths ranging between approximately 3.0 and 11.6 feet below the ground surface, where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. The bedrock consisted of soft to hard, slightly weathered to decomposed, slightly to highly fractured sandstone. Severely decomposed siltstone was encountered in boring B-15 above the sandstone. The amount of rock recovered in each core run varied between 97 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 23 and 100 percent with an average of 75 percent indicating fair to good rock.

Unconfined compressive strength of tested cores ranged between 11,775 psi and 13,299 psi. The tested cores correspond to samples at depths between 5.7 feet and 22.0 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-15	5.7-6.1	140.1	12,960
B-15	14.1-14.5	148.2	13,299
B-16	19.1-19.5	146.0	11,775
B-16	21.6-22.0	142.3	13,040
B-17	14.6-15.0	135.6	12,292
B-17	19.6-20.0	150.0	12,114

4.2.3 Groundwater Conditions

Seepage was not observed in any of the borings drilled for this structure. There were no measurable water levels in the borings prior to rock coring. Measurable water levels were present in borings TR-43 through TR-45 upon the completion of coring between approximate depths of 2.0 and 6.7 feet. Water was used during rock coring and masked any seepage zones that might exist in the rock.

It should be noted that groundwater levels may fluctuate with seasonal variations and following periods of heavy or prolonged precipitation, and therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be needed to obtain a more accurate estimate of the groundwater table elevation.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is anticipated that the existing bridge will be constructed as described in Sections 1 and 2 of this report. It is understood through comments from ODOT's Office of Structural Engineering that spread footings will be used to support the abutments. However, the use of drilled shafts and pipe piles have also been considered to support the abutments. Additionally, the site is well suited for the use of MSE walls to contain the abutments and hold back the roadway embankment. Recommendations for the piles, drilled shafts, spread footings, and MSE walls are presented in the following sections.

5.1 Bridge Foundation Recommendations

5.1.1 Rear and Forward Abutments

It is understood through discussions with TranSystems that spread footings are preferred to support the abutments. Due to the small amount of settlement that is anticipated, assuming a well constructed MSE wall, spread footings are well suited to support the abutments. As per the Bridge Design Manual 204.6.2.1, an allowable bearing capacity of 4 ksf may be used to design the footings. The MSE walls as recommended will be founded on bedrock or granular fill placed on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be negligible.

It is understood through previous comments from the ODOT Office of Structural Engineering (OSE) that pipe piles may be considered to support the abutments. It is understood that the abutments could be supported by steel pipe piles placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel pipe pile in the prebored hole, grout or cement should be placed in the annular area around the pile in the prebored hole prior to constructing the embankment / MSE wall (per OSE). Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment and MSE wall construction. The allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

Due to the relatively small rigidity of the steel pipe piles compared to drilled shafts, the steel pipe piles are anticipated to provide little resistance to lateral earth pressures that can be induced in high embankment fills such as those at the proposed structure. Therefore, the prebored and socketed steel pipe pile foundation system may be a concern if significant lateral loads are present.

As mentioned above, drilled shafts have also been considered for the support of the abutments. Given the site conditions, it appears that drilled shafts socketed a minimum of 5 feet into competent rock will be well suited for the support of the proposed structural abutments. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

It is recommended that skin friction in the overburden soil/fill and shallow rock socket be neglected. The bearing surface should be clean and free of loose material and water prior to placement of concrete. The drilled center-to-center spacing of drilled shafts should generally be no less than 2.5 times their diameter. A qualified representative of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance ignoring any end bearing.

The drilled shaft design parameters cited above consider axial loading only. If it is necessary to design drilled shafts to resist significant lateral loads, DLZ should be informed of the loading conditions to ensure recommendations for adequate socket lengths are provided.

When using drilled shaft foundations in conjunction with MSE retaining walls, it is necessary to consider the placement of the drilled shafts with respect to the MSE wall and soil reinforcing straps. Drilled shafts should be installed at a sufficient distance from the back of the MSE wall such that the soil reinforcement can be splayed around the shafts with splay angles of 15 degrees or less. From the center of the drilled shafts to the back of the MSE wall, this dimension is approximately two times the shaft diameter.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although the borings did not encounter significant seepage, water could flow into the drilled shafts during installation particularly below the stream level and within wet zones that may be present in the rock or soil. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

Table 2 summarizes the site conditions and foundation recommendations. It should be noted that the bedrock surface varies slightly across the project area. The approximate bearing elevations presented below indicate the elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 2-Summary of Foundation Recommendations

Structural Element	Structure / Boring	Existing Ground Surface Elevation + (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-15	551.8	Pipe Piles	542.3 *	Pile Capacity++
			Drilled Shafts	542.3 *	80 ksf+++
			Spread Footings	MSE Fill**	4 ksf
	Right / TR-44	556.7	Pipe Piles	540.7 *	Pile Capacity++
			Drilled Shafts	540.7 *	80 ksf+++
			Spread Footings	MSE Fill**	4 ksf
Forward Abutment	Left / B-16	556.8	Pipe Piles	538.6 *	Pile Capacity++
			Drilled Shafts	538.6 *	80 ksf+++
			Spread Footings	MSE Fill**	4 ksf
	Right / B-17	558.1	Pipe Piles	543.5 *	Pile Capacity++
			Drilled Shafts	543.5 *	80 ksf+++
			Spread Footings	MSE Fill**	4 ksf

* Includes 5-foot socket into competent rock.

** Bearing elevation should be determined by a qualified engineer as the foundation alternative is selected.

+ Surveyed ground surface elevation at the boring.

++ Pile capacity should conform to ODOT BDM 202.2.3.2.

+++ End bearing capacity only.

5.2 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls would be used to construct the embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. The MSE wall should be constructed per the recommendations presented in this report and in conformance with the manufacturer's specifications and common practice.

5.2.1 MSE Retaining Walls: General Information

An MSE retaining wall essentially consists of good quality backfill material with layers of metal or plastic reinforcing that are attached to concrete facing panels. The MSE wall and associated backfill should be constructed in accordance with the specifications of the manufacturer of the MSE wall.

At the time this report was prepared, it was understood that spread footings would be used at this site to support the bridge abutments. However, the MSE walls at this site have been evaluated using both spread footings and deep foundations. If the foundation type should change, DLZ should be informed so that the analyses may be revised as necessary.

The MSE walls were analyzed for bearing capacity, sliding, overturning, global stability, and settlement. Calculations are presented in Appendix IV. Other external and internal stability analyses (ie. reinforcing strap design) are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented in Table 3. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. However, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 3-Soil Parameters Used in MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Rock (Rear Abutment)	Bedrock	145	NA	NA	NA	NA
Foundation Soil (Forward Abutment)	Native	125	1750	0	0	29
Foundation Soil (Rear Abutment)	Native	125	2250	0	0	29
Foundation Soil (Forward/Rear Abutments)	Compacted Granular Fill	120	0	34	0	34

5.2.2 MSE Retaining Wall Evaluations and Recommendations

Rear and Forward Abutment MSE Retaining Walls

Based on the structure site plan, the maximum height of the MSE wall at the rear abutment (station 61+70.5 is approximately 39.5 feet. The overburden in the area of the rear abutment is relatively thin (4.0 to 11.0 feet). Similarly, the maximum height of the MSE wall at the forward abutment (station 62+62.5) is approximately 32.0 feet. The overburden in the area of the forward abutment is also relatively thin (9.6 to 12.0 feet).

The soil profile encountered at the forward abutment (boring B-16) location was found to be the most critical at this site. Consequently, global stability analyses and settlement calculations are based upon the results of borings drilled for the forward abutment location. Bearing capacity and stability (sliding and overturning) calculations are presented for both the rear and the forward abutment locations using both spread footings and deep foundations to support the bridge abutments.

Initially, analyses were undertaken to ascertain the global stability, bearing capacity and stability (sliding and overturning) of the MSE walls bearing on the native / existing soils. The results of the analyses indicated that the factors of safety for global stability, sliding, overturning, and drained bearing capacity were adequate. However, bearing capacity calculations indicated that the factor of safety for the undrained bearing capacity is below the recommended minimum value of 2.5.

To address the low undrained bearing capacity, it is recommended that the relatively shallow existing soils be excavated to bedrock and replaced with compacted granular fill to the leveling pad elevation. The compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. Alternatively, the leveling pad may be placed on the top of bedrock. If founded on bedrock, no embedment into the rock is required. The limits of the "remove and replace" area should extend beyond the edge of the MSE wall/select granular footprint by the depth of the aggregate base as per ODOT BDM Figure 330. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to ODOT BDM Item 204. In addition, because the wall will be founded on or near bedrock, global stability should be adequate. For stability, calculations have shown that a minimum reinforcement length of the full height (H+D) times 0.70 must be used for the proposed MSE walls at this location, using a compacted granular fill foundation.

It should be noted that variations in the topography will likely be encountered within the proposed footprint of the proposed MSE wall, causing the top of bedrock elevation to vary significantly. In areas where compacted granular fill is to be placed on bedrock, a level bench must be cut into the rock to place the fill for stability purposes.

An alternative to undercutting the existing soils to bedrock is to construct the MSE wall at the rear and forward abutments using staged construction. Due to the low undrained shear strength of the existing soils, analyses have indicated that a maximum allowable stage of 22 feet may be constructed between consolidation waiting periods while maintaining adequate undrained factors of safety.

Using staged construction, for MSE walls founded on the existing soils, calculations have shown that a minimum reinforcement length of the full height (H+D) times 0.95 must be used for the proposed MSE walls at this location, using spread footings to support the abutments. Similarly, calculations have shown that a minimum reinforcement length of the full height (H+D) times 0.80 must be used for the proposed MSE walls at this location, using deep foundations to support the abutments.

Settlement calculations have been performed for the MSE retaining wall at the forward abutment location, and are considered representative of both the rear and forward abutments. The total maximum settlement of the MSE wall volumes at the forward abutment was estimated to be approximately 1 inch at the centerline of the wall. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankments. Differential settlement at this location was negligible. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1/100). Additionally, time-rate of settlement calculations have indicated that approximately 238 days will be required to achieve 90 percent consolidation of the foundation soils. Settlement calculations are presented in Appendix IV.

Due to the inherent variations of the subsurface conditions, the actual required waiting period may be shorter or longer than anticipated. It is recommended that piezometers be installed in the clay layer to monitor the excess pore water pressures that will develop during construction and ensure that a critical pore water pressure is not exceeded. Analyses have been performed to determine the critical pore water pressures. Based upon the results of the analyses, if the water level in the piezometer rises 14.0 feet above the existing ground surface, construction should halt immediately. Construction may continue after pore pressures in the clay layer have dissipated. The results of the critical pore pressure stability analyses are presented in Appendix IV.

It should be noted that the MSE wall at the rear abutment is in close proximity to a drainage channel, which is essentially parallel to State Route 140 (Webster St.). A 48-inch culvert is currently proposed to route the water in front of the MSE wall. The approximate elevation of bedrock under the MSE wall at the rear abutment ranges from 548.8 to 548.7, which is near the bottom of the drainage channel elevation. If scour and erosion near the toe of the MSE wall are of concern, then the leveling pad of the MSE wall should be founded on bedrock and slope protection should be provided with riprap.

Calculations for bearing capacity, overturning and sliding are attached for the native soil and compacted granular fill foundations. Global stability analyses performed for MSE walls bearing on the native soils have indicated adequate undrained and drained global stability.

On the following pages, tables 4 through 7 present the results of the MSE retaining wall stability calculations for both the rear and forward abutments, using deep foundations and spread footings to support the abutments. Additional information regarding the results of calculations and parameters required for the design of MSE retaining walls is included in Appendix V.

Table 4- Results of MSE Retaining Wall Analyses				
Rear Abutment Location (Using Deep Foundations to support Structure)				
No Undercut	Global Stability	Undrained	FS>1.5[+]	
		Drained	FS>1.5[+]	
		Drained-Seismic	FS>1.5[+]	
	Bearing Capacity	Undrained	**FS=1.78****	q_a=4,695 psf
		Drained	FS=2.86	q_a=7,526 psf
	Stability	Sliding	FS=1.58	
		Overturning	FS=5.29	
Undercut to Bedrock*	Global Stability[++]	Undrained	FS>1.5	
		Drained	FS>1.5	
		Drained-Seismic	FS>1.3	
	Bearing Capacity	Undrained	FS=4.75	q_a=13,321 psf
		Drained	FS=4.75	q_a=13,321 psf
	Stability	Sliding	FS=1.80	
		Overturning	FS=4.15	

* Analyses assume that native soil is undercut to the top of rock and the MSE wall leveling pad is constructed on compacted granular fill, which is placed on bedrock.

** Inadequate factor of safety. Factor of safety should be >2.5.

[+] Forward abutment wall location was determined to be the most critical at this site. Refer to the forward abutment for analysis information.

Table 5- Results of MSE Retaining Wall Analyses				
Rear Abutment Location (Using Spread Footing Foundations to support Structure)				
No Undercut	Global Stability	Undrained	FS>1.5[+]	
		Drained	FS>1.5[+]	
		Drained-Seismic	FS>1.5[+]	
	Bearing Capacity	Undrained	**FS=1.49****	q_a=4,695 psf
		Drained	FS=2.68	q_a=8,426 psf
	Stability	Sliding	FS=1.85	
		Overturning	FS=7.25	
Undercut to Bedrock*	Global Stability[++]	Undrained	FS>1.5	
		Drained	FS>1.5	
		Drained-Seismic	FS>1.3	
	Bearing Capacity	Undrained	FS=3.45	q_a=12,583 psf
		Drained	FS=3.45	q_a=12,583 psf
	Stability	Sliding	FS=1.80	
		Overturning	FS=4.15	

* Analyses assume that native soil is undercut to the top of rock and the MSE wall leveling pad is constructed on compacted granular fill, which is placed on bedrock.

** Inadequate factor of safety. Factor of safety should be >2.5.

[+] Forward abutment wall location was determined to be the most critical at this site. Refer to the forward abutment for analysis information.

[++] Assumes that MSE wall is founded on bedrock or on granular fill near bedrock, global stability is assumed to be adequate.

Table 6- Results of MSE Retaining Wall Analyses				
Forward Abutment Location (Using Deep Foundations to support Structure)				
No Undercut	Global Stability	Undrained	FS=1.9	
		Drained	FS=1.6	
		Drained-Seismic	FS=1.5	
	Bearing Capacity	Undrained	**FS=1.67****	q_a=3,667 psf
		Drained	FS=2.82	q_a=6,185 psf
	Stability	Sliding	FS=1.52	
		Overturning	FS=4.97	
Undercut to Bedrock*	Global Stability[++]	Undrained	FS>1.5	
		Drained	FS>1.5	
		Drained-Seismic	FS>1.3	
	Bearing Capacity	Undrained	FS=4.73	q_a=11,032 psf
		Drained	FS=4.73	q_a=11,032 psf
	Stability	Sliding	FS=1.75	
		Overturning	FS=3.96	

* Analyses assume that native soil is undercut to the top of rock and the MSE wall leveling pad is constructed on compacted granular fill, which is placed on bedrock.
** Inadequate factor of safety. Factor of safety should be >2.5.

Table 7- Results of MSE Retaining Wall Analyses				
Forward Abutment Location (Using Spread Footing Foundations to support Structure)				
No Undercut	Global Stability	Undrained	FS=1.9	
		Drained	FS=1.9	
		Drained-Seismic	FS=1.8	
	Bearing Capacity	Undrained	**FS=1.31****	q_a=3,667 psf
		Drained	FS=2.54	q_a=7,103 psf
	Stability	Sliding	FS=1.85	
		Overturning	FS=7.32	
Undercut to Bedrock*	Global Stability[++]	Undrained	FS>1.5	
		Drained	FS>1.5	
		Drained-Seismic	FS>1.3	
	Bearing Capacity	Undrained	FS=3.24	q_a=10,568 psf
		Drained	FS=3.24	q_a=10,568 psf
	Stability	Sliding	FS=1.75	
		Overturning	FS=3.96	

* Analyses assume that native soil is undercut to the top of rock and the MSE wall leveling pad is constructed on compacted granular fill, which is placed on bedrock.
** Inadequate factor of safety. Factor of safety should be >2.5.
[++] Assumes that MSE wall is founded on bedrock or on granular fill near bedrock, global stability is assumed to be adequate.

5.3 Groundwater Considerations

Water seepage was not observed in any of the borings. Groundwater was not noted prior to adding drill water. Representative final water levels (including core water) could not be obtained due to the use of water during rock coring. Foundation construction on the rock is expected to encounter only minor to moderate seepage due to the proximity of a stream. Excavations or shafts extending below ground level may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage and water flow that may enter any excavations.

6.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning our report.

Respectfully submitted,

DLZ OHIO, INC.

Steven Riedy
Geotechnical Engineer

(SJR)

Wael Alkasawneh, P.E.
Geotechnical Engineer

sjr

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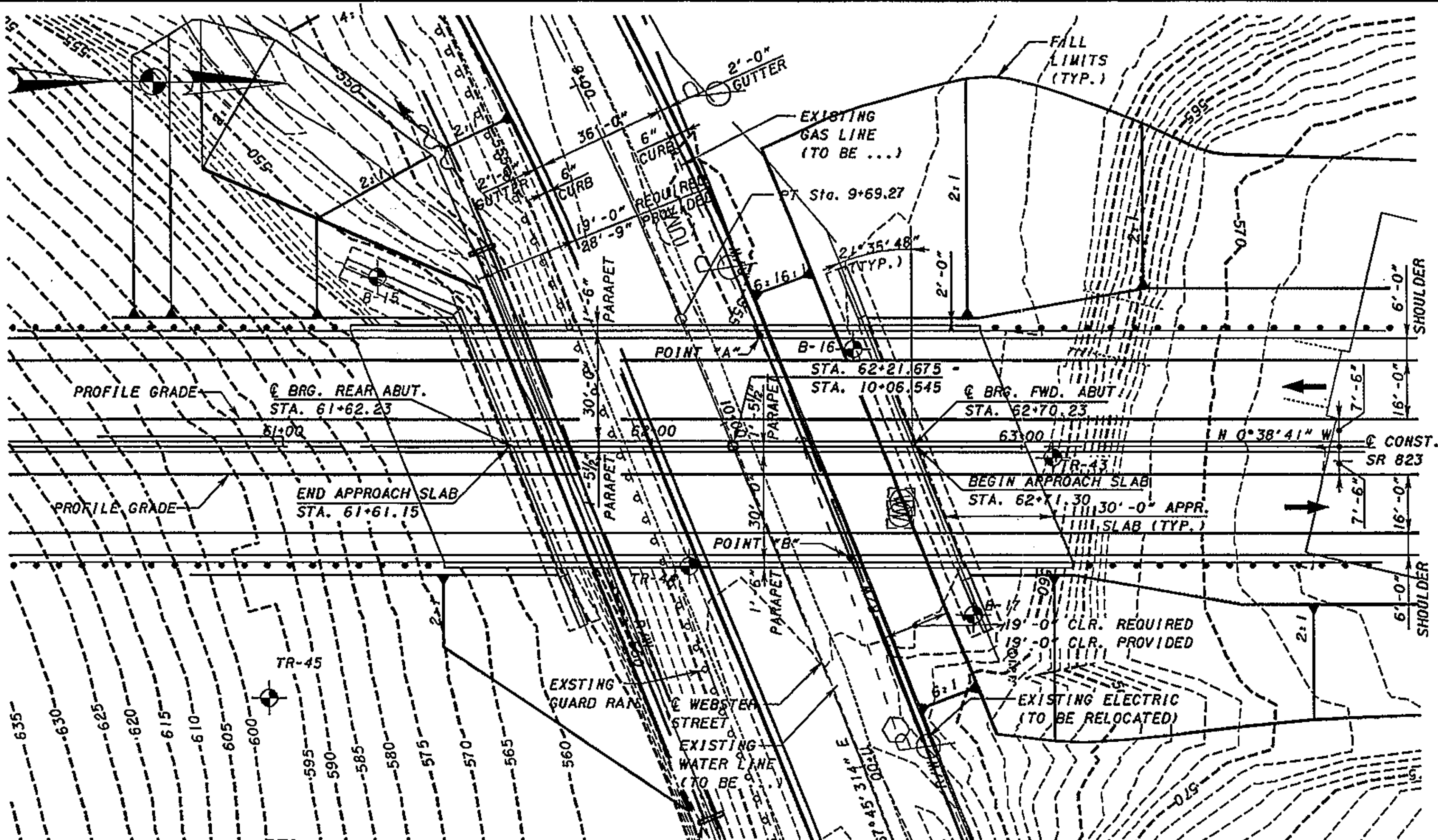
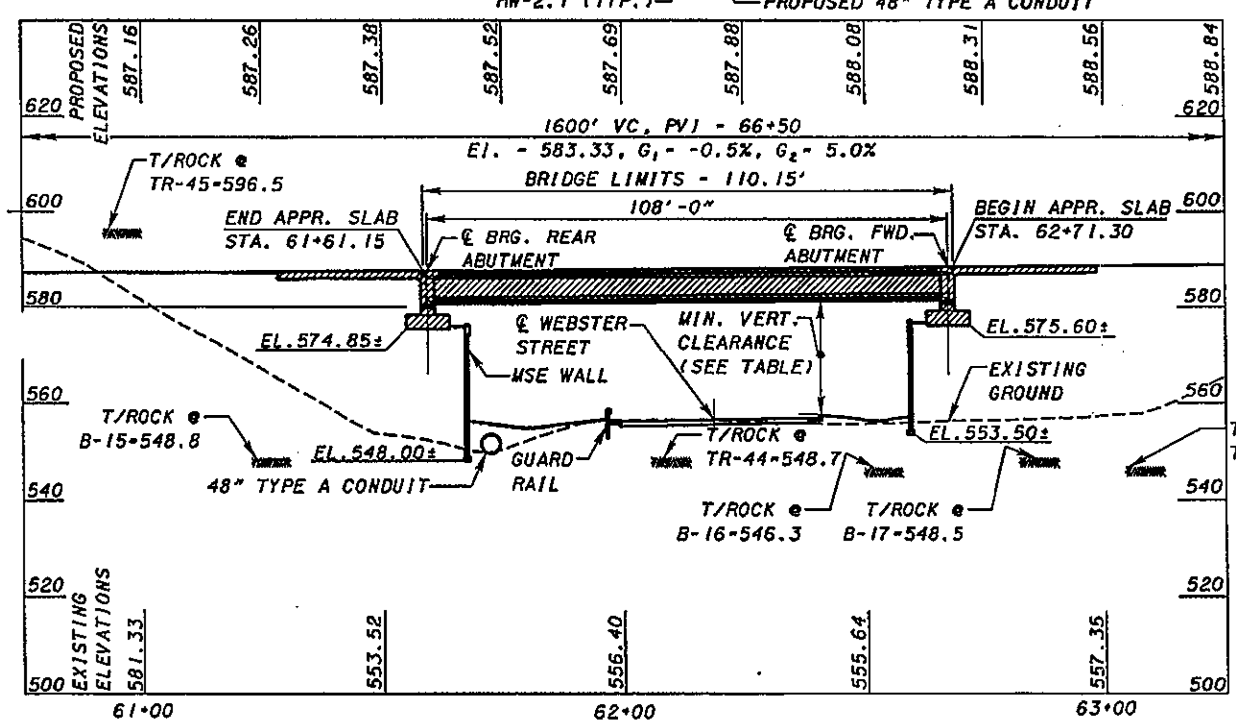
APPENDIX I

Structure Plan and Profile Drawing – 11"x17"
Boring Plan – 11"x17"



PLAN



PROFILE ALONG PROFILE GRADE S.R. 823 LEFT BRIDGE

TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	24.09'	23.66'
PREFERRED	17.0'	17.0'

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	OFFSET
REAR ABUT.	61+39.34	31.50 RT.
REAR ABUT.	61+14.41	31.50 LT.
FWD. ABUT.	63+18.09	31.50 RT.
FWD. ABUT.	62+93.15	31.50 LT.

BENCHMARK 1	BENCHMARK 2
(TO BE PROVIDED LATER)	(TO BE PROVIDED LATER)

TRAFFIC DATA

S.R. 823
CURRENT YEAR ADT (2010) - 13,400
DESIGN YEAR ADT (2030) - 21,000
CURRENT YEAR ADTT (2010) - 1,876
DESIGN YEAR ADTT (2030) - 2,940

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
3. FOR CULVERT DETAILS SEE SHEET

FOUNDATION DATA:

ALLOWABLE BEARING CAPACITY ON MSE WALL EMBANKMENT 4 KSF.

LEGEND

BTA-1 - BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 - BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ - BORING LOCATION

PROPOSED STRUCTURE

TYPE: SINGLE SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAM WITH COMPSITE REINFORCED CONCRETE DECK SUPPORTED BY SEMI-INTEGRAL ABUTMENTS AND MSE WALL SUPPORTED EMBANKMENT.
SPANS: 108'-0" C/C BEARING
ROADWAY: 2 - 30'-0" T/T PARAPETS
LOADING: HS-25 AND ALTERNATE MILITARY LOADING, FWS-60 PSF
SKEW: 21°35'48" RF
CROWN: NORMAL, 0.016 FT/FT
ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30'-0" LONG)
LATITUDE: 82° 52' 23" N
LONGITUDE: 38° 45' 14" W

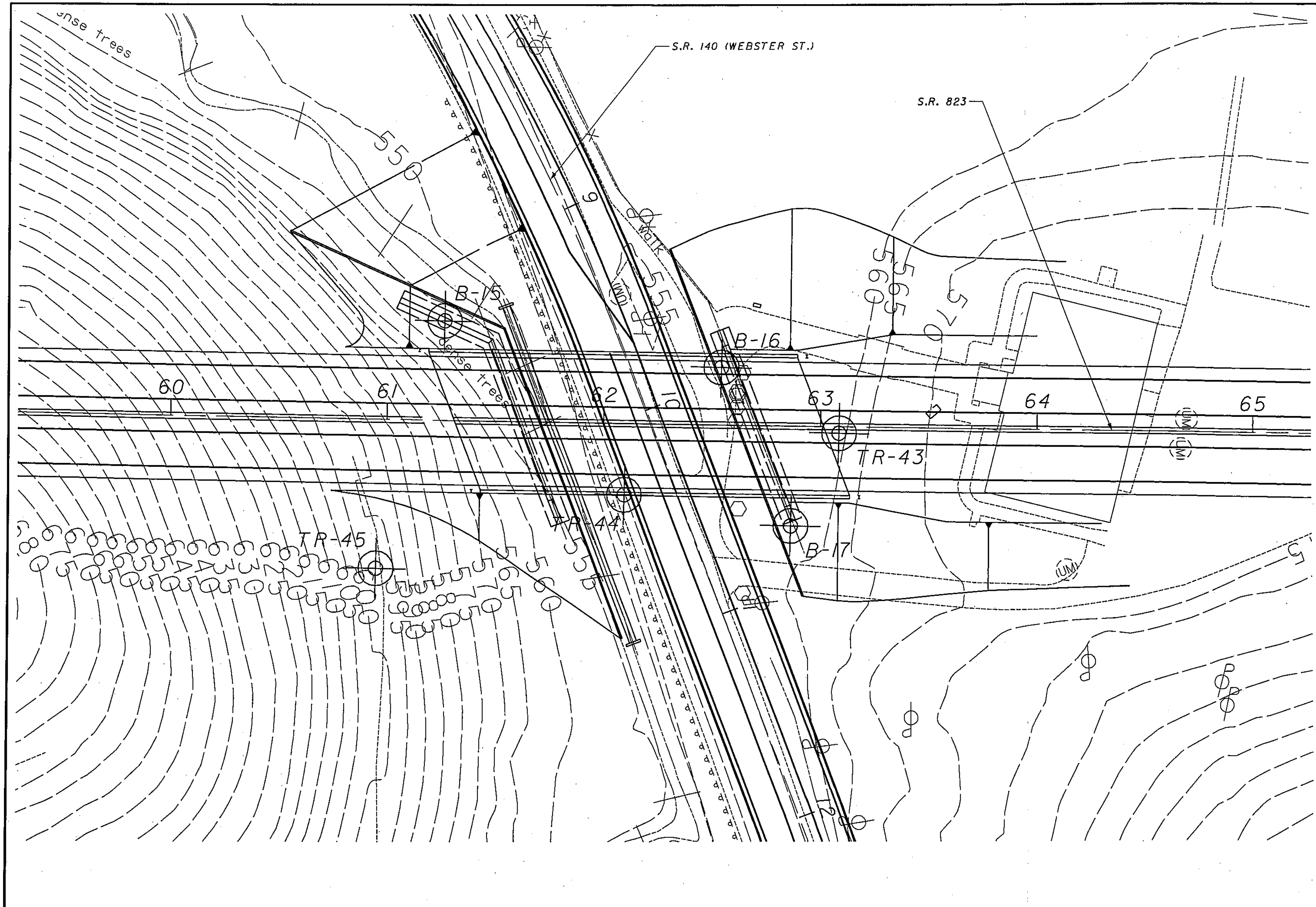
DESIGN AGENCY: Systems
DESIGNED PJP | CHECKED MSL | DRAWN CAS | REVISED MTN | REVIEWED JRC | DATE 05/31/06 | STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 61+61.15
STA. 62+71.30

SITE PLAN
BRIDGE NO. SCI-823-0117
S.R. 823 OVER WEBSTER STREET (SR 140)

SCI-823-0.00
PID 77366

1/2



HORIZONTAL SCALE IN FEET
0 10 20 40

CALCULATED
CHECKED

BORING PLAN
S.R. 823 OVER S.R. 140 (WEBSTER ST.)

SCI-823

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Six (6) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

Drilling and sampling were conducted in accordance with procedures generally recognized and accepted as standardized methods of investigation of subsurface conditions concerning geotechnical engineering considerations. Borings were drilled with either a truck-mounted or ATV-mounted drill rig.

Drive split-barrel sampling was performed in 1.5 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

In borings where rock was cored, NXM or NQ size diamond coring tools were used.

In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses and plasticity characteristics tests. The results of these tests are shown on the boring logs.

The boring logs included in the Appendix have been prepared on the basis of the field record of drilling and sampling, and the results of the laboratory examination and testing of samples. Stratification lines on the boring logs indicating changes in soil stratigraphy represent depths of changes approximated by the driller, by sampling effort and recovery, and by laboratory test results. Actual depths to changes may differ somewhat from the estimated depths, or transitions may occur gradually and not be sharply defined. The boring logs presented in this report therefore contain both factual and interpretative information and are not an exact copy of the field log.

Although it is considered that the borings have disclosed information generally representative of site conditions, it should be expected that between borings conditions may occur which are not precisely represented by any one of the borings. Soil deposition processes and natural geologic forces are such that soil and rock types and conditions may change in short vertical intervals and horizontal distances.

Soil/rock samples will be stored at our laboratory for a period of six months. After this period of time, they will be discarded, unless notified to the contrary by the client.

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LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

1. Depth (in feet) – refers to distance below the ground surface.

2. Elevation (in feet) – is referenced to mean sea level, unless otherwise noted.

3. Standard Penetration (N) – the number of blows required to drive a 2-inch O.D., 1-3/8 inch I.D., split-barrel sampler, using a 140-pound hammer with a 30-inch free fall. The blows are recorded in 6-inch drive increments. Standard penetration resistance is determined from the total number of blows required for one foot of penetration by summing the second and third 6-inch increments of an 18-inch drive.

   50/n – indicates number of blows (50) to drive a split-barrel sampler a certain number of inches (n) other than the normal 6-inch increment.

4. The length of the sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.

5. Sample recovery from each drive is indicated numerically in the column headed "Recovery".

6. The drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.

7. The length of hydraulically pressed "Undisturbed" samples is indicated graphically by horizontal lines across the "Press" column.

8. Sample numbers are designated consecutively, increasing in depth.

9. Soil Description

   a. The following terms are used to describe the relative compactness and consistency of soils:

   Granular Soils – Compactness

Term	Blows/Foot Standard Penetration
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	over 50

   Cohesive Soils – Consistency

Term	Unconfined Compression tons/sq.ft.	Blows/Foot Standard Penetration	Hand Manipulation
Very Soft	less than 0.25	below 2	Easily penetrated by fist
Soft	0.25 – 0.50	2 – 4	Easily penetrated by thumb
Medium Stiff	0.50 – 1.0	4 – 8	Penetrated by thumb with moderate pressure
Stiff	1.0 – 2.0	8 – 15	Readily indented by thumb but not penetrated
Very Stiff	2.0 – 4.0	15 – 30	Readily indented by thumb nail
Hard	over 4.0	over 30	Indented with difficulty by thumb nail

   b. Color – If a soil is a uniform color throughout, the term is single, modified by such adjective as light and dark. If the predominant color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled".

   c. Texture is based on the Ohio Department of Transportation Classification System. Soil particle size definitions are as follows:

Description	Size	Description	Size
Boulders	Larger than 8"	Sand – Coarse	2.0 mm to 0.42 mm
Cobbles	8" to 3"	– Fine	0.42 mm to 0.074 mm
Gravel – Coarse	3" to ¾"	Silt	0.074 mm to 0.005 mm
– Fine	¾" to 2.0 mm	Clay	smaller than 0.005 mm

d. The main soil component is listed first. The minor components are listed in order of decreasing percentage of particle size.

e. Modifiers to main soil descriptions are indicated as a percentage by weight of particle sizes.

trace	0 to 10%
little	10 to 20%
some	20 to 35%
"and"	35 to 50%

f. Moisture content of **cohesionless soils** (sands and gravels) is described as follows:

Term	Relative Moisture or Appearance
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Wet	Voids filled with free water

g. The moisture content of **cohesive soils** (silts and clays) is expressed relative to plastic properties.

Term	Relative Moisture or Appearance
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit but below liquid limit
Wet	Moisture content above liquid limit

10. Rock Hardness and Rock Quality Designation

a. The following terms are used to describe the relative hardness of the **bedrock**.

Term	Description
Very Soft	Permits denting by moderate pressure of the fingers. Resembles hard soil but has rock structure. (Crushes under pressure of fingers and/or thumb)
Soft	Resists denting by fingers, but can be abraded and pierced to shallow depth by a pencil point. (Crushes under pressure of pressed hammer)
Medium Hard	Resists pencil point, but can be scratched with a knife blade. (Breaks easily under single hammer blow, but with crumbly edges.)
Hard	Can be deformed or broken by light to moderate hammer blows. (Breaks under one or two strong hammer blow, but with resistant sharp edges.)
Very Hard	Can be broken only by heavy and in some rocks repeated hammer blows.

b. Rock Quality Designation, RQD – This value is expressed in percent and is an indirect measure of rock soundness. It is obtained by summing the total length of all core pieces which are at least four inches long, and then dividing this sum by the total length of the core run.

11. Gradation – when tests are performed, the percentage of each particle size is listed in the appropriate column (defined in Item 9c).

12. When a test is performed to determine the natural moisture content, liquid limit moisture content, or plastic limit moisture content, the moisture content is indicated graphically.

13. The standard penetration (N) value in blows per foot is indicated graphically.

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Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-15 | Location: Sta. 61+26.0, 45.9 ft. LT of SR 823 CL | Date Drilled: 9/20/06

WATER OBSERVATIONS: Water seepage at: None observed
Water level at completion: None (prior to adding core water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○
	551.8													
0.2	551.6						Topsoil - 2"							
		4 9 15	10	1		4.5+	Hard brown SILT (A-4b), little clay, trace to little fine sand, trace to little coarse sand, little gravel; dry to damp.	12	9	--	9	56	15	Non-Plastic
3.0	548.8	50/2	2	2		-	Severely decomposed brown SILTSTONE, arenaceous.							50+
4.5	547.3	Core 30"	Rec 29"	RQD 87%	R-1		Medium hard to hard gray SANDSTONE, fine to medium grained, moderately to slightly weathered, medium to thickly bedded, slightly fractured. @ 4.5' to 4.7', brown. @ 4.9', 5.1', 5.2', 8.1', argillaceous, low angle fractures. @ 5.7' to 6.1', qu=12,960 psi, Er=2,626,964 psi. @ 8.6' to 8.8', high angle fracture, brown.							
		Core 60"	Rec 60"	RQD 97%	R-2									
10.7	541.1						Medium hard to hard gray SANDSTONE interbedded with SILTSTONE; very fine to fine grained, moderately weathered, medium bedded, highly to moderately fractured. @ 11.4', 11.9', 13.3', 13.6', argillaceous, low angle fractures. @ 12.1' to 12.3', high angle fracture. @ 14.1' to 14.5', qu=13,299 psi.							
		Core 30"	Rec 30"	RQD 80%	R-3									
14.5	537.3						Bottom of Boring - 14.5'							

Depth scale: 5, 10, 15, 20, 25, 30

SPT scale: 10, 20, 30, 40

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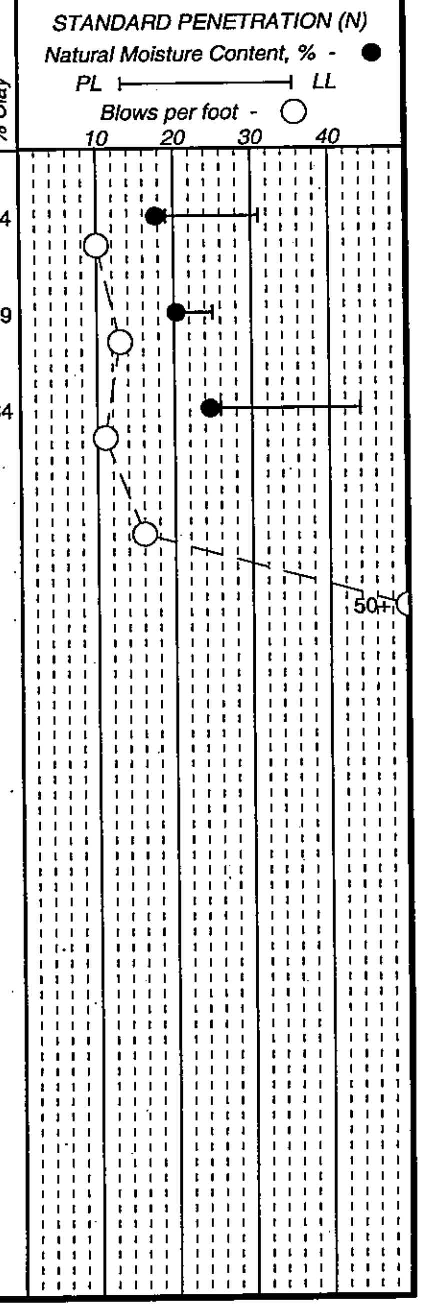
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-16 | Location: Sta. 62+54.1, 26.5 ft. LT of SR 823 CL | Date Drilled: 9/19/06

WATER OBSERVATIONS: Water seepage at: None observed
Water level at completion: None (prior to adding core water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	556.8	8 6 4	12	1		3.0	Very stiff brown SILT AND CLAY (A-6a), trace fine sand, trace coarse sand, trace gravel; damp to moist.	5	1	--	6	65	24
3.0	553.8	3 4 9	15	2		3.25	Very stiff brown SILT (A-4b), trace fine sand, trace coarse sand, little clay; damp to moist.	0	1	--	7	73	19
5.5	551.3	3 4 7	18	3		3.5	Very stiff mottled brown and gray CLAY (A-7-6), trace fine sand, trace gravel; damp to moist.	1	0	--	1	34	64
		5 7 9	16	4		3.5	@ 8.0', little coarse sand.						
10.5	546.3	23 50/5	10	5		-	Severely weathered brown SANDSTONE, argillaceous.						
12.0	544.8	Core 60"	Rec 60"	RQD 23%	R-1		Medium hard brown SANDSTONE; fine to medium grained, highly weathered, broken. @ 13.2', highly fractured, clay/silt filled low angle fractures.						
14.4	542.4	Core 60"	Rec 59"	RQD 73%	R-2		Medium hard to hard gray SANDSTONE; fine to medium grained, moderately weathered, thinly bedded, highly fractured. @ 14.6', 14.9', 15.1', 15.4', 15.7', low angle, iron stained fractures. @ 17.5', 17.7', 17.9', low angle, clay filled fractures. @ 18.8', moderately fractured. @ 19.1' to 19.5', qu=11,775 psi, Er=2,364,092 psi. @ 21.6' to 22.0', qu=13,040 psi.						
22.0	534.8						Bottom of Boring - 22.0'						

Depth scale: 0, 5, 10, 15, 20, 25, 30



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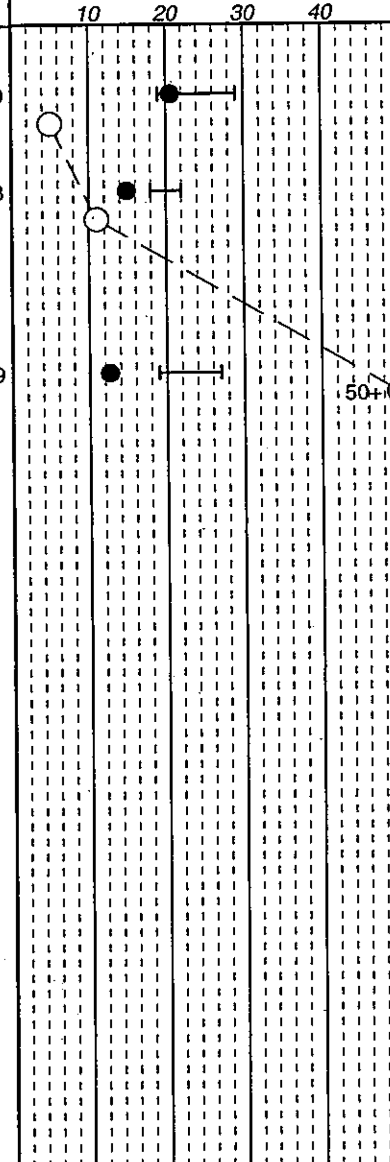
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-17 | *Location:* Sta. 62+86.7, 45.9 ft. RT of SR 823 CL | *Date Drilled:* 9/19/06

WATER OBSERVATIONS: Water seepage at: None observed
Water level at completion: None (prior to adding core water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.1	1, 3, 2	12	1		2.0	Stiff to very stiff SILT (A-4b), trace fine sand, trace coarse sand, trace gravel; moist.	4	1	--	4	63	29
3.0	555.1	4, 5, 6	18	2		-	Medium dense brown SANDY SILT (A-4a), trace to little fine sand, little clay, little coarse sand, little gravel; damp.	18	12	--	12	45	13
7.0	551.1	6, 42, 50/1	12	3		2.75	Very stiff brown, SILT (A-4b), trace to little fine sand, trace to little coarse sand, trace to little gravel; damp.	10	10	--	9	52	19
9.6	548.5						Medium hard brown SANDSTONE interbedded with SILTSTONE; very fine to fine grained, highly weathered, argillaceous, highly fractured to broken.						
11.5	546.6	Core 60"	Rec 59"	RQD 37%	R-1		Medium hard to hard gray SANDSTONE; fine to medium grained, moderately weathered, thickly bedded, highly fractured. @ 11.5' to 13.1', iron stained, high angle fractures. @ 14.2', moderately fractured. @ 14.6' to 15.0', qu=12,292 psi, Er=2,406,830 psi.						
16.0	542.1	Core 60"	Rec 59"	RQD 67%	R-2		Medium hard to hard gray SANDSTONE interbedded with SILTSTONE; very fine to fine grained, moderately weathered, medium bedded, highly fractured. @ 18.7', 18.9', 19.3', clay/silt filled low angle fractures. @ 19.6' to 20.0', qu=12,114 psi.						
20.0	538.1						Bottom of Boring - 20.0'						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



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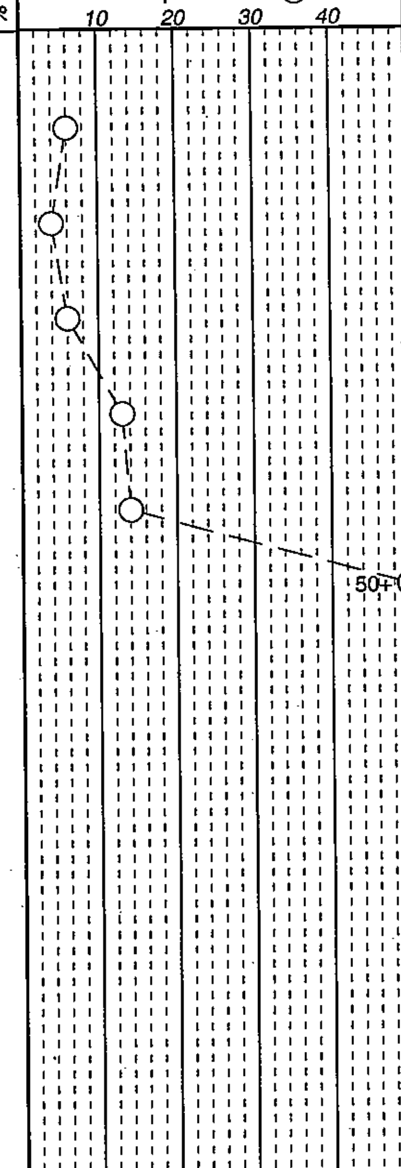
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-43 | Location: Sta. 63+08.6, 2.9 ft. RT of SR 823 CL | Date Drilled: 02/03/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 2.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	558.4	2 3 3	15	1		2.25	Very stiff brown SILT AND CLAY (A-6a), trace fine sand; damp to moist.						
		2 2 2	16	2		1.75	@ 3.5 to 5.5', stiff to very stiff, brown and gray.						
5.5	552.9	2 3 3	17	3		2.0	Stiff to very stiff brown and gray SILTY CLAY (A-6b), little fine sand, trace fine gravel; moist.						
		3 5 8	18	4		1.75							
11.6	546.8	5 8 6	14	5			Severely weathered brown SANDSTONE argillaceous.						
		15 50/5	10	6									
15.0	543.4	Core 120"	Rec 120"	RQD 78%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, highly weathered, argillaceous, micaceous, massively bedded, highly fractured, with typical low and high angle clay filled and rust stained fractures. @ 20.1' to 20.4', ferric band. @ 20.5', argillaceous lamination.						
21.0	537.4						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, massively bedded, unfractured to slightly fractured.						
		Core 120"	Rec 120"	RQD 100%	R-2								

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○



Depth scale: 0, 5, 10, 20, 25, 30

FILE: 0121-3070-03 [1/5/2007 4:19 PM]

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Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-43 | Location: Sta. 63+08.6, 2.9 ft. RT of SR 823 CL | Date Drilled: 02/03/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 2.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	528.4													
30.3	528.1						Medium hard to hard gray SANDSTONE; very fine grained, slightly weathered, argillaceous, micaceous, thinly bedded, unfractured to slightly fractured.							
35.0	523.4						Bottom of Boring - 35.0'							

Depth scale: 40, 45, 50, 55, 60

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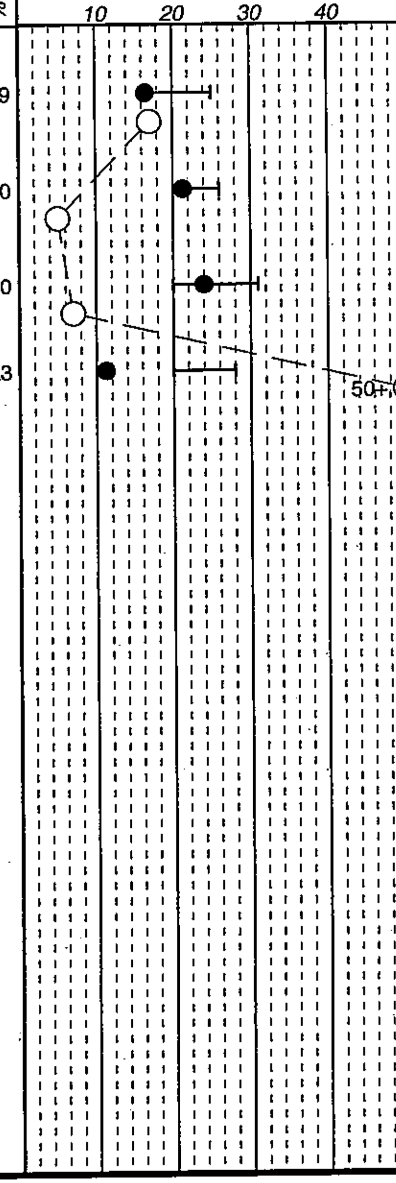
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-44 | Location: Sta. 62+09.9, 32.7 ft. RT of SR 823 CL | Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 5.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	556.7						Asphalt Concrete Pavement - 12"						
1.0	555.7	9 8 9	16	1		4.0	Hard brown and gray SILT (A-4b), some clay, trace fine to coarse sand; damp.	1	2	--	6	62	29
3.0	553.7	3 2 3	13	2			Loose brown SANDY SILT (A-4a), some gravel, little clay; damp.	22	12	--	12	44	10
5.5	551.2	3 3 4	14	3		2.25	Very stiff brown and gray SILT AND CLAY (A-6a), trace fine sand; moist.	0	3	--	6	61	30
8.0	548.7	26 50/3	8	4			Severely weathered brown SANDSTONE argillaceous, micaceous.	0	1	--	6	70	23
11.0	545.7						Medium hard gray SANDSTONE; very fine to fine grained, highly weathered, argillaceous, micaceous, massively bedded, highly fractured, with typical low angle rust stained fractures.						
12.4	544.3	Core 108"	Rec 108"	RQD 73%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, massively bedded, unfractured to slightly fractured.						
							@ 17.7' to 18.0', broken zone, clay filled.						
							@ 19.0' to 20.0', high angle fractures.						
20		Core 120"	Rec 116"	RQD 92%	R-2		@ 24.2' to 24.6', ferric band.						
30.0	526.7						Bottom of Boring - 30.0'						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL |———| LL
Blows per foot - ○



FILE: 0121-3070-03 [1/5/2007 4:19 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc.
Project: SCI-823-0.00
Job No. 0121-3070.03

LOG OF: Boring TR-44
Location: Sta. 62+09.9, 32.7 ft. RT of SR 823 CL
Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 5.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	526.7													
35														
40														
45														
50														
55														
60														

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-45 | Location: Sta. 60+95.8, 68.5 ft. RT of SR 823 CL | Date Drilled: 02/24/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 6.7' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ (10 20 30 40)
	596.6													
0.1	596.5						Topsoil - 2"							
		15 50/4	10	1			Severely weathered brown SANDSTONE argillaceous.							50+
		50/3	3	2										50+
5.0	591.6						@ 5.0'-5.1', broken zones.							
		Core 120"	Rec 120"	RQD 79%	R-1		Soft to medium hard brownish gray SANDSTONE; very fine grained, highly weathered to decomposed, argillaceous, micaceous, massively bedded, highly fractured, with typical low angle rust stained and clay filled fractures. @ 7.4' to 7.6', broken zones. @ 9.3' to 9.6', high angle rust stained fracture. @ 11.1' to 11.4', broken zones. @ 14.2'-14.5', high angle rust stained fracture.							
15.4	581.2	Core 120"	Rec 120"	RQD 100%	R-2		Medium hard to hard gray SANDSTONE; very fine grained, slightly weathered, argillaceous, micaceous, massively bedded, slightly fractured. @ 16.1'-16.3', rust stained zone. @ 17.3', low angle clay filled fracture. @ 20.2', low angle clay filled fracture.							
25.0	571.6						Bottom of Boring - 25.0'							

Depth scale: 0, 5, 10, 15, 20, 25, 30

FILE: 0121-3070-03 [1/5/2007 4:19 PM]

APPENDIX III

Laboratory Test Results

Unconfined Compression of Rock Core Specimens
(ASTM D-2938)

DLZ Project No.: 0121-3070.03 **Client: TranSystems**

Project Name: SCI-823-0.00 **Date:** 12/19/2006

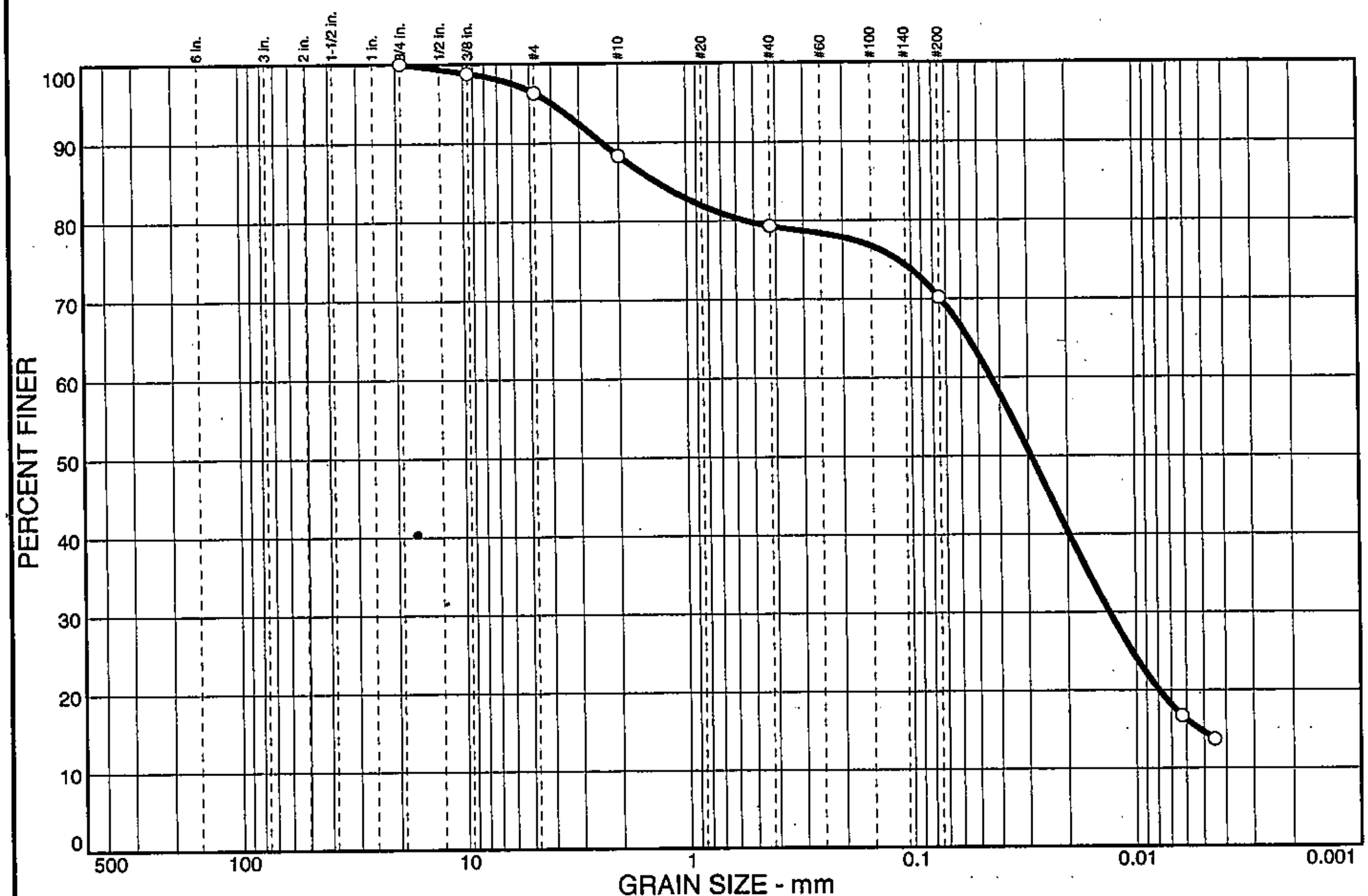
Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-15	R-1	5.7-6.1	1.977	1.975	1.975	1.974	4.741	4.742	4.741	4.741	2.402	0.0083948	533.72	140.16	39,670	**12,960**
			1.974	1.970	1.974											
B-15	R-3	14.1-14.5	1.974	1.979	1.976	1.977	4.627	4.627	4.626	4.627	2.341	0.0082125	552.52	148.32	40,810	**13,299**
			1.977	1.977	1.977											
B-16	R-2	19.1-19.5	1.976	1.979	1.977	1.978	4.174	4.174	4.177	4.175	2.111	0.0074221	491.86	146.10	36190	**11,775**
			1.977	1.981	1.979											
B-16	R-2	21.6-22.0	1.983	1.983	1.982	1.982	4.524	4.526	4.526	4.525	2.283	0.0080774	521.71	142.39	40,240	**13,040**
			1.982	1.982	1.981											
B-17	R-1	14.6-15.0	1.978	1.976	1.978	1.977	4.378	4.375	4.378	4.377	2.214	0.0077733	478.41	135.68	37,740	**12,292**
			1.978	1.976	1.977											
B-17	R-2	19.6-20.0	1.981	1.984	1.981	1.980	4.601	4.604	4.599	4.601	2.324	0.0081951	557.7	150.03	37300	**12,114**
			1.976	1.980	1.978											

DLZ

Engineers * Architects * Scientists

6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	3.7	8.0	9.0	9.1	55.6	14.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.8		
#4	96.3		
#10	88.3		
#40	79.3		
#200	70.2		

* (no specification provided)

Soil Description

Silt with sand

Atterberg Limits

PL= NP LL= NP PI= NP

Coefficients

D_{85}= 1.36 D_{60}= 0.0441 D_{50}= 0.0292
D_{30}= 0.0132 D_{15}= 0.0053 D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

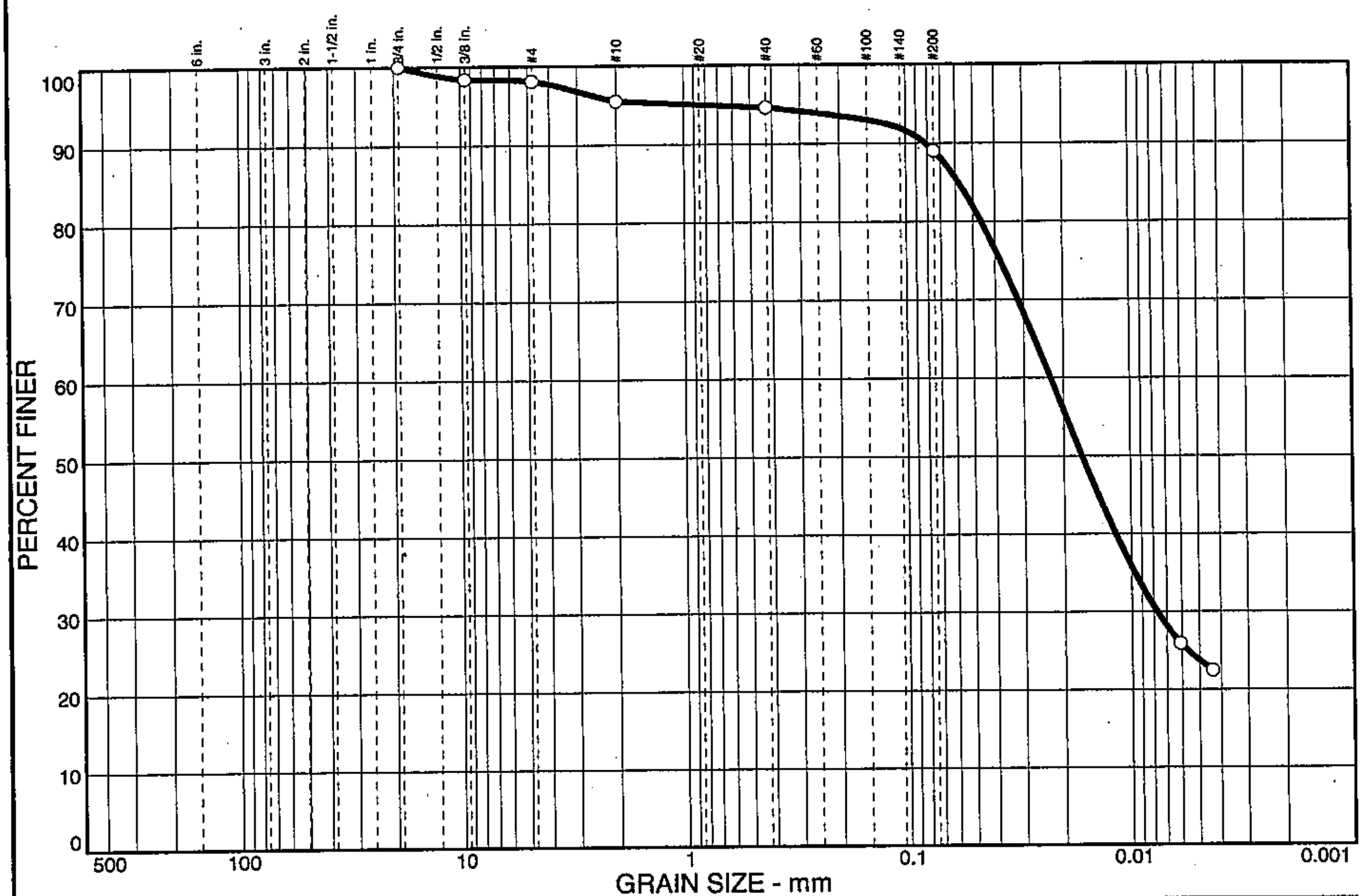
Moisture Content= 6.2%

Sample No.: 1 **Source of Sample:** B-15 **Date:** 12/19/06
Location: **Elev./Depth:** 1.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	1.9	2.6	0.9	5.6	65.3	23.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.4		
#4	98.1		
#10	95.5		
#40	94.6		
#200	89.0		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 31 PI= 12

Coefficients

D_{85}= 0.0584 D_{60}= 0.0229 D_{50}= 0.0166
D_{30}= 0.0078 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(10)

Remarks

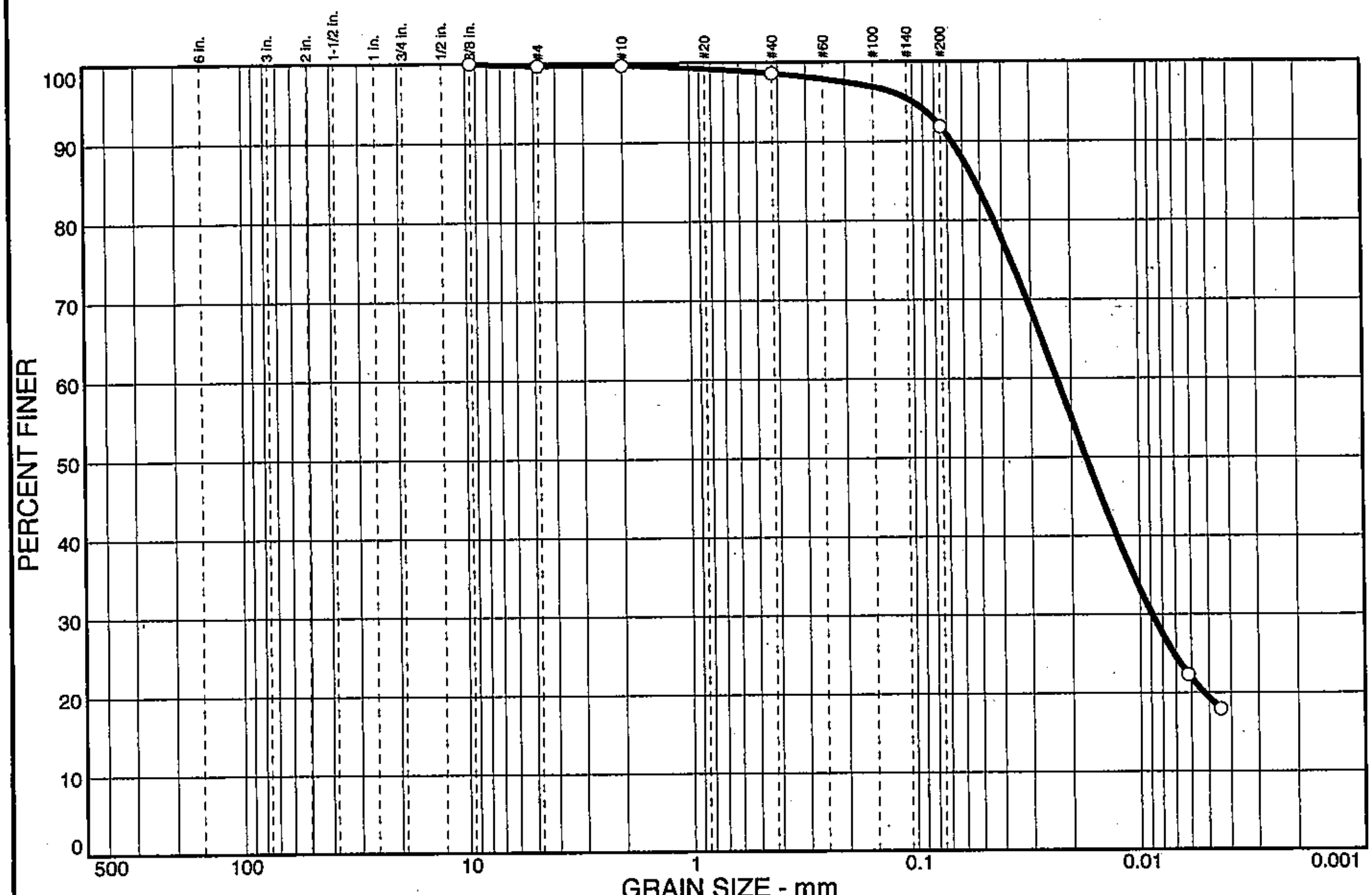
Moisture Content= 17.7%

Sample No.: 1 **Source of Sample:** B-16 **Date:** 12/19/06
Location: **Elev./Depth:** 1.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.3	0.0	1.1	6.8	72.5	19.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.7		
#10	99.7		
#40	98.6		
#200	91.8		

* (no specification provided)

Soil Description

Silty clay

Atterberg Limits

PL= 20 LL= 25 PI= 5

Coefficients

D_{85}= 0.0530 D_{60}= 0.0232 D_{50}= 0.0172
D_{30}= 0.0089 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(3)

Remarks

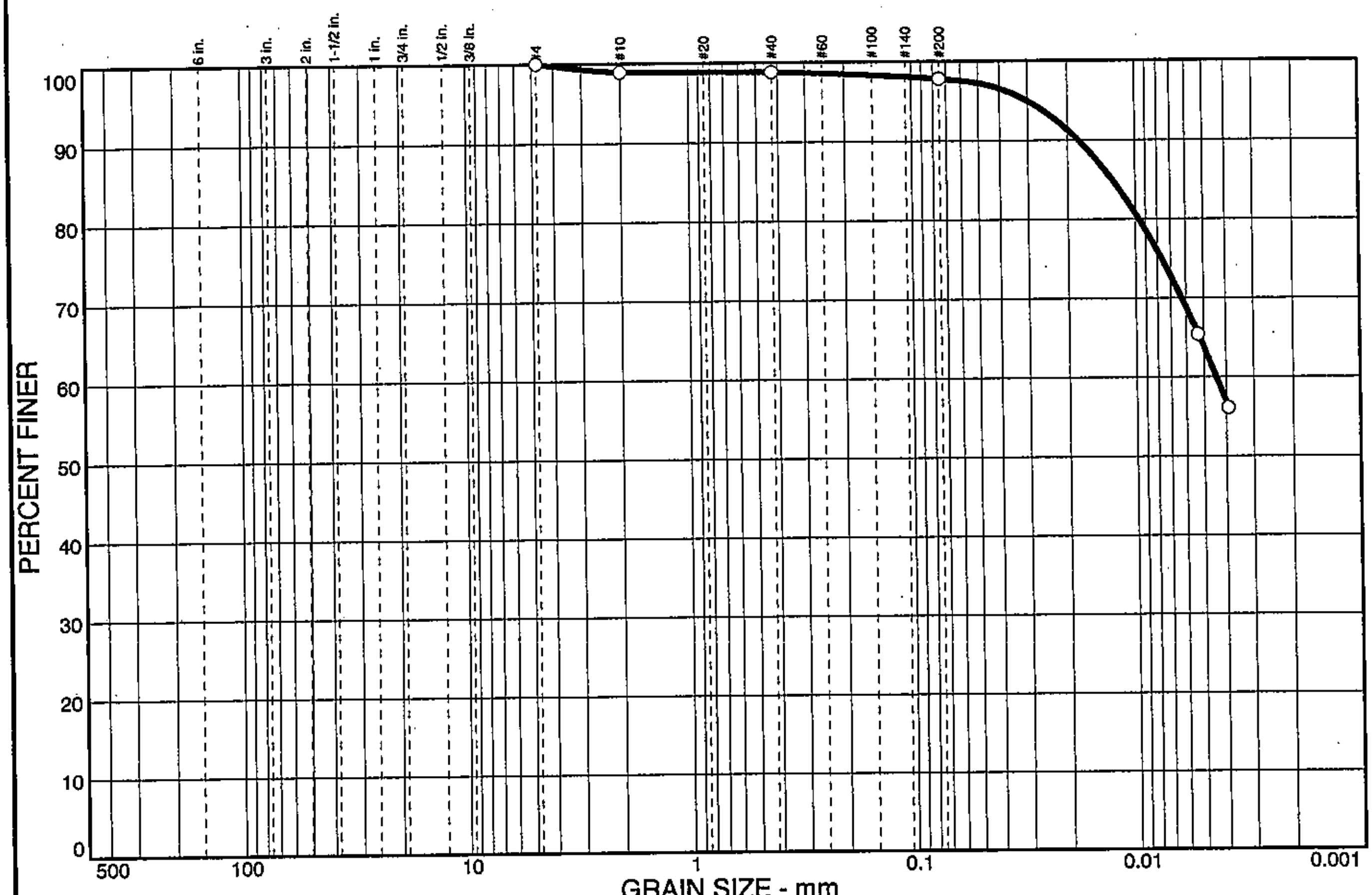
Moisture Content= 20.3%

Sample No.: 2 **Source of Sample:** B-16 **Date:** 12/19/06
Location: **Elev./Depth:** 3.5

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	1.1	0.1	1.0	34.2	63.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.9		
#40	98.8		
#200	97.8		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 26 LL= 44 PI= 18

Coefficients

D_{85}= 0.0127 D_{60}= 0.0044 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(20)

Remarks

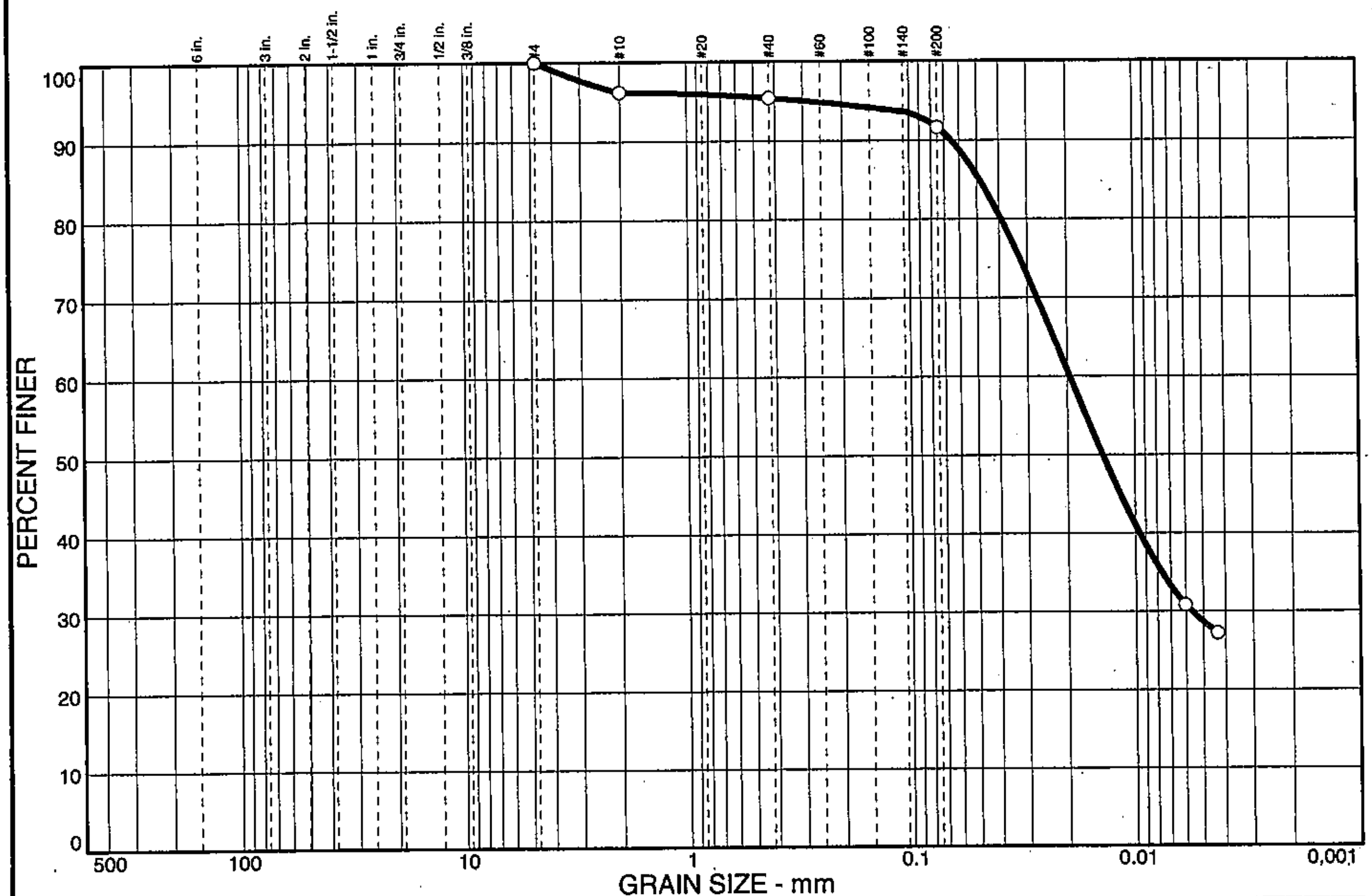
Moisture Content= 24.6%

Sample No.: 3 **Source of Sample:** B-16 **Date:** 12/19/06
Location: **Elev./Depth:** 6.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	3.8	0.8	3.8	63.0	28.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	96.2		
#40	95.4		
#200	91.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 29 PI= 10

Coefficients

D_{85}= 0.0490 D_{60}= 0.0195 D_{50}= 0.0140
D_{30}= 0.0057 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(8)

Remarks

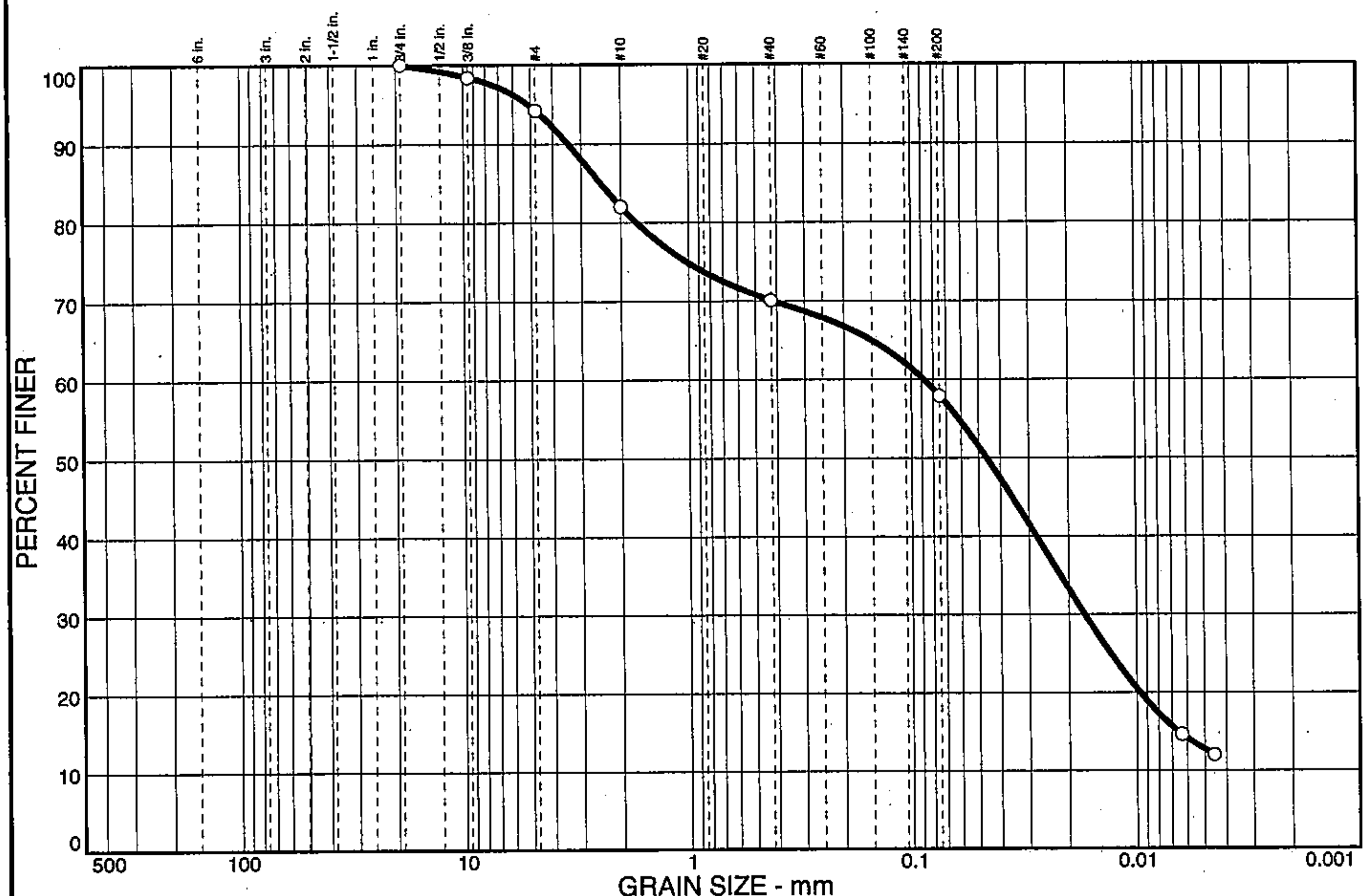
Moisture Content= 20.6%

Sample No.: 1 **Source of Sample:** B-17 **Date:** 12/19/06
Location: **Elev./Depth:** 1.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	5.8	12.2	11.9	12.2	45.2	12.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	98.4		
#4	94.2		
#10	82.0		
#40	70.1		
#200	57.9		

* (no specification provided)

Soil Description

Sandy silty clay

Atterberg Limits

PL= 18 LL= 22 PI= 4

Coefficients

D_{85}= 2.46 D_{60}= 0.0883 D_{50}= 0.0463
D_{30}= 0.0171 D_{15}= 0.0066 D_{10}=
C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(0)

Remarks

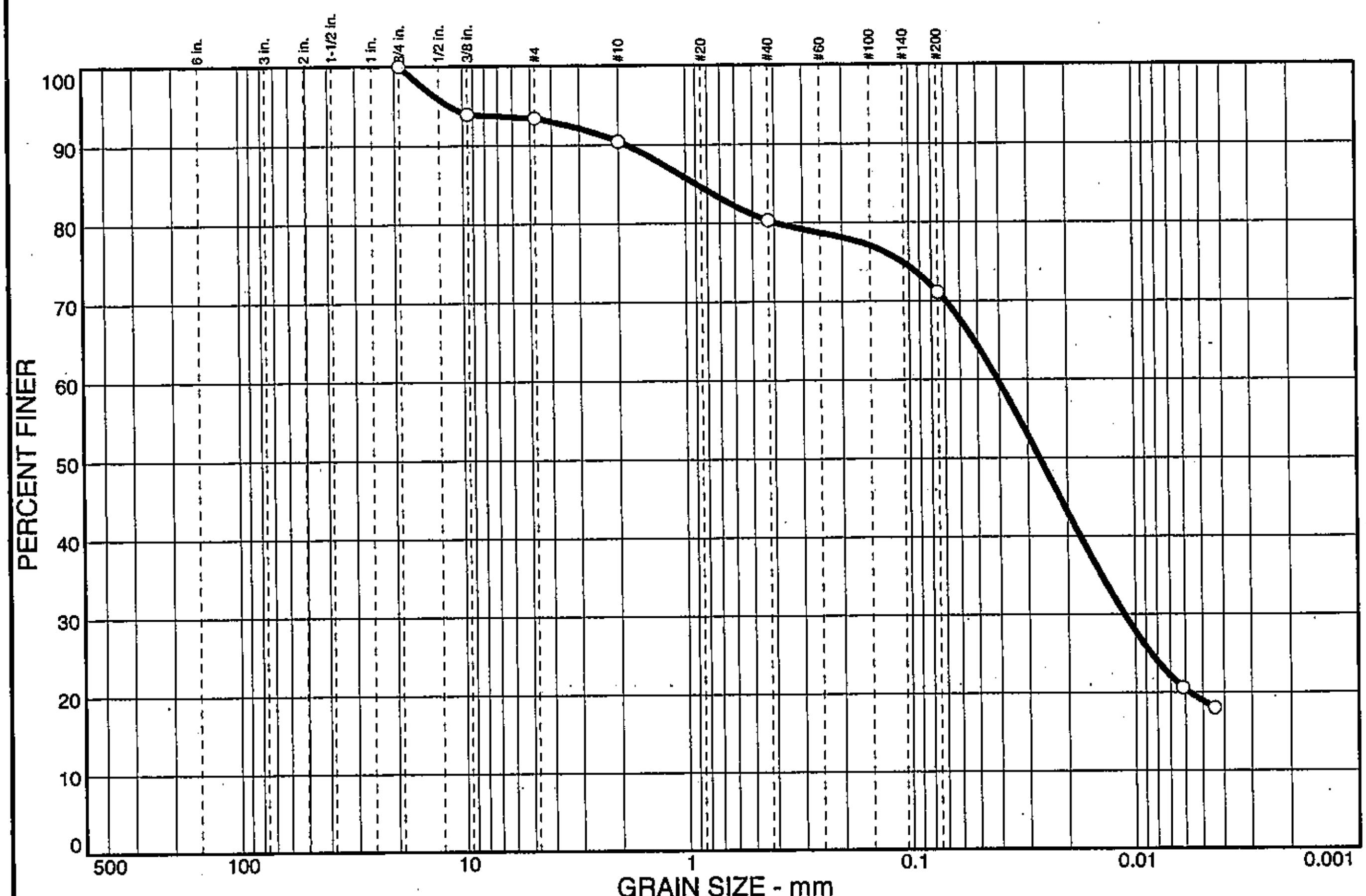
Moisture Content= 14.9%

Sample No.: 2 **Source of Sample:** B-17 **Date:** 12/19/06
Location: **Elev./Depth:** 3.5

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	6.6	3.0	10.1	9.2	52.2	18.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	93.9		
#4	93.4		
#10	90.4		
#40	80.3		
#200	71.1		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 19 LL= 27 PI= 8

Coefficients

D_{85}= 0.902 D_{60}= 0.0409 D_{50}= 0.0267
D_{30}= 0.0112 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(4)

Remarks

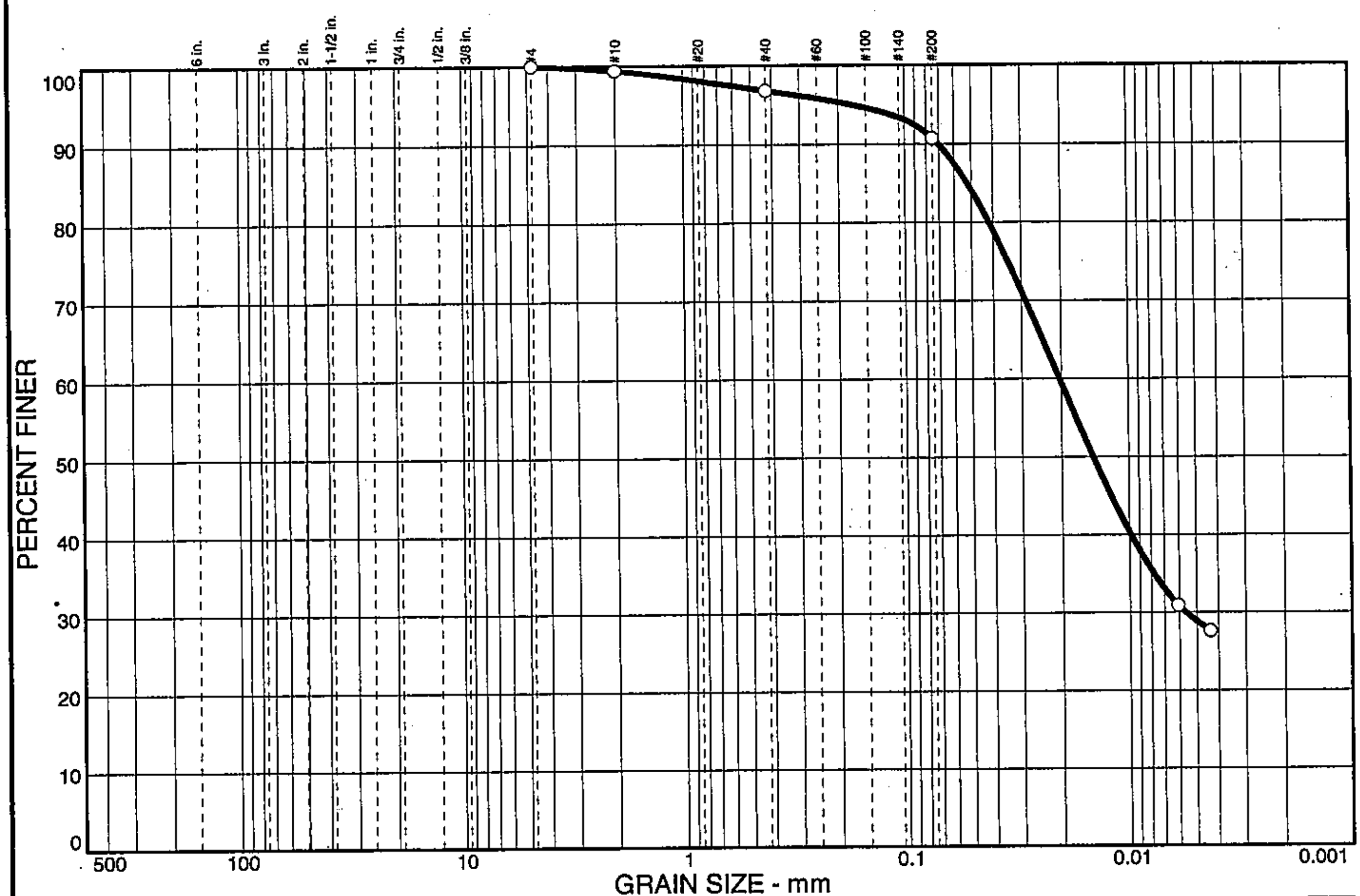
Moisture Content= 12.6%

Sample No.: 3 **Source of Sample:** B-17 **Date:** 12/19/06
Location: **Elev./Depth:** 8.5

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.6	2.6	6.1	61.9	28.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.4		
#40	96.8		
#200	90.7		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 17 LL= 25 PI= 8

Coefficients

D_{85}= 0.0526 D_{60}= 0.0205 D_{50}= 0.0145
D_{30}= 0.0056 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

Remarks

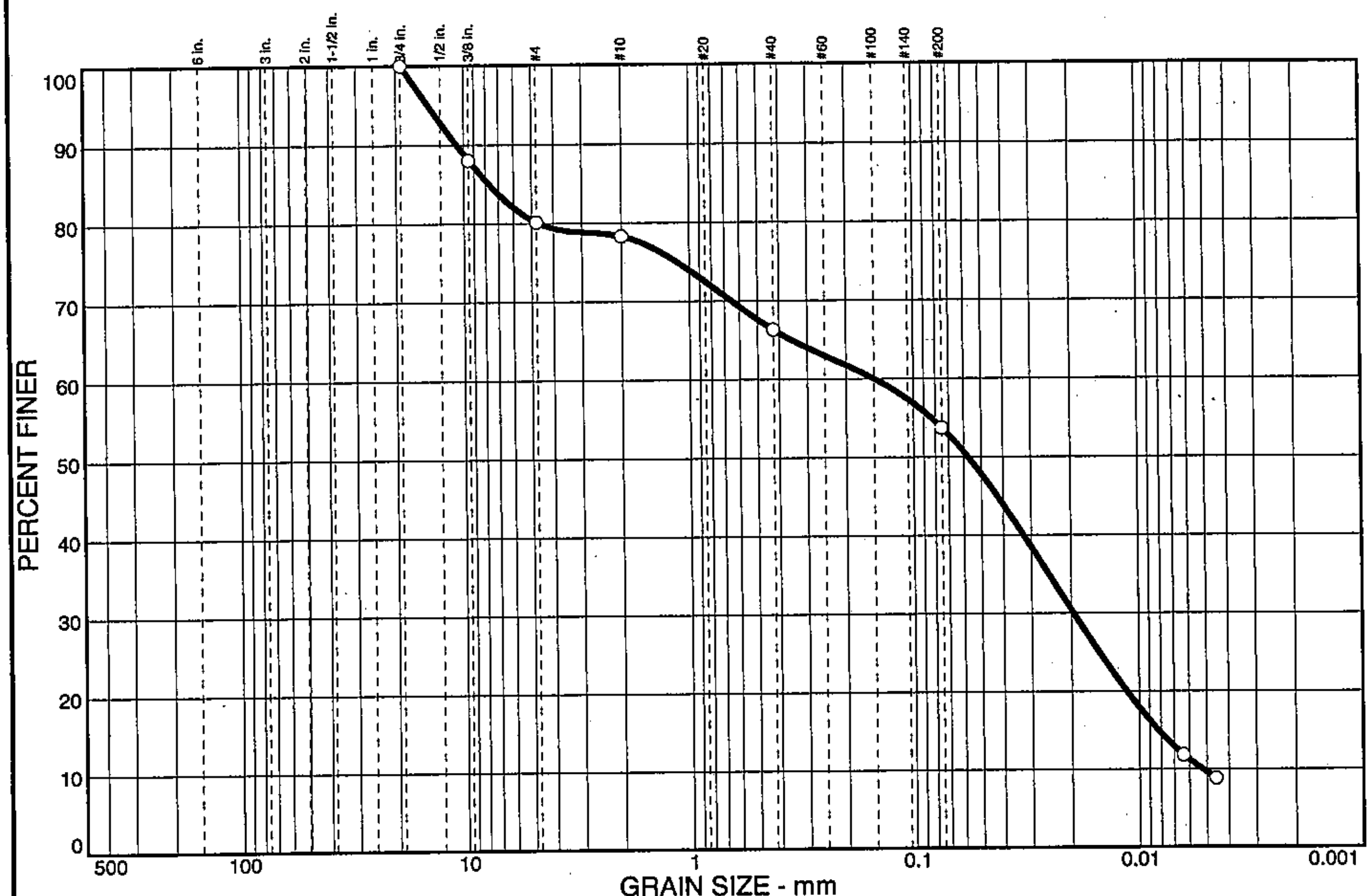
Moisture Content= 16.5%

Sample No.: 1 **Source of Sample:** TR-44 **Date:** 3/29/05
Location: **Elev./Depth:** 1.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	19.9	1.8	12.0	12.6	44.1	9.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	88.0		
#4	80.1		
#10	78.3		
#40	66.3		
#200	53.7		

* (no specification provided)

Soil Description

Sandy silt with gravel

Atterberg Limits

PL= 22 LL= 26 PI= 4

Coefficients

D_{85}= 7.75 D_{60}= 0.150 D_{50}= 0.0576
D_{30}= 0.0196 D_{15}= 0.0083 D_{10}= 0.0053
C_u= 28.51 C_c= 0.49

Classification

USCS= ML AASHTO= A-4(0)

Remarks

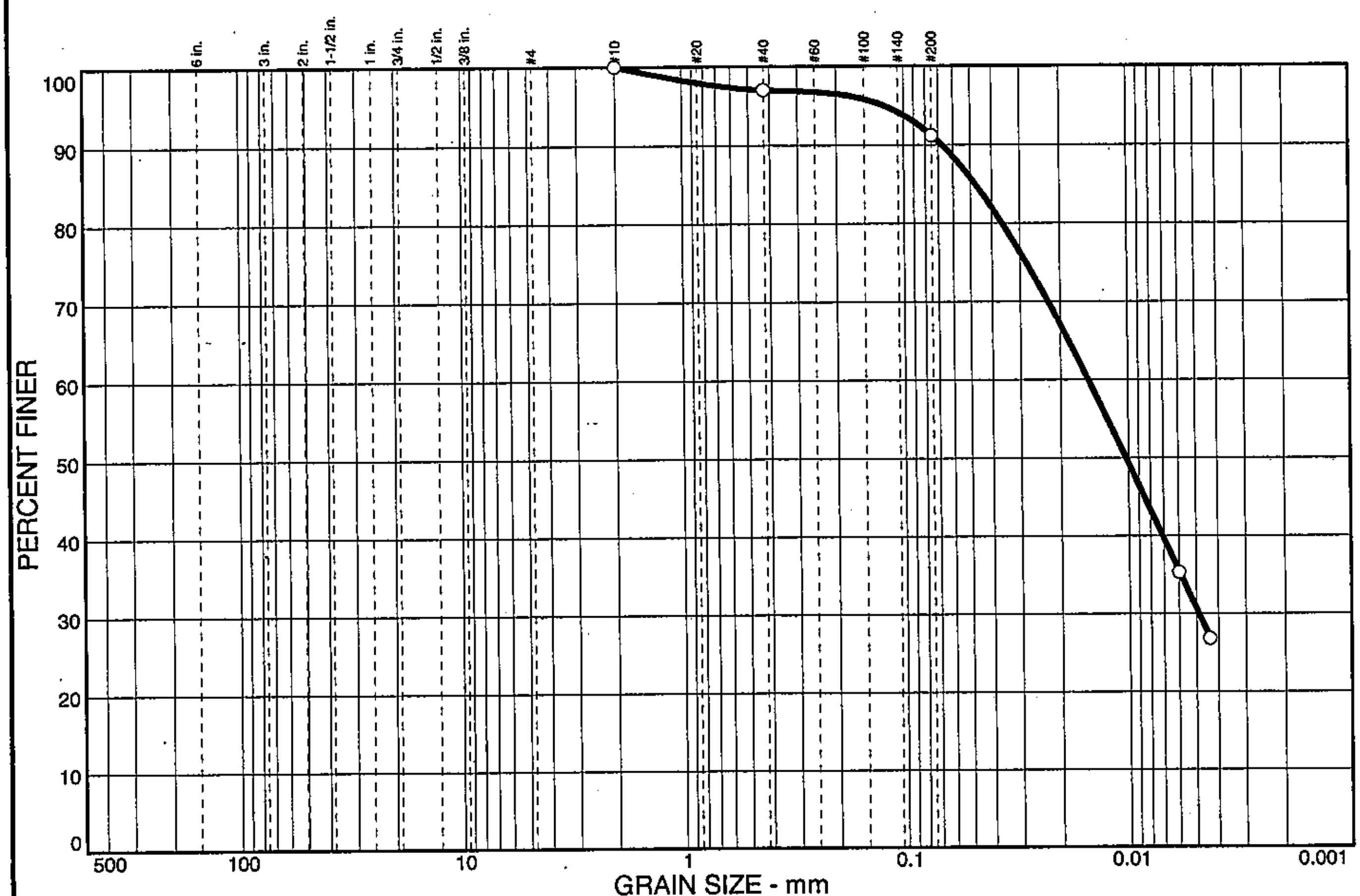
Moisture Content= 21.3%

Sample No.: 2 **Source of Sample:** TR-44 **Date:** 3/29/05
Location: **Elev./Depth:** 3.5

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	3.0	5.8	60.7	30.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	97.0		
#200	91.2		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 31 PI= 11

Coefficients

D_{85}= 0.0478 D_{60}= 0.0151 D_{50}= 0.0103
D_{30}= 0.0049 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

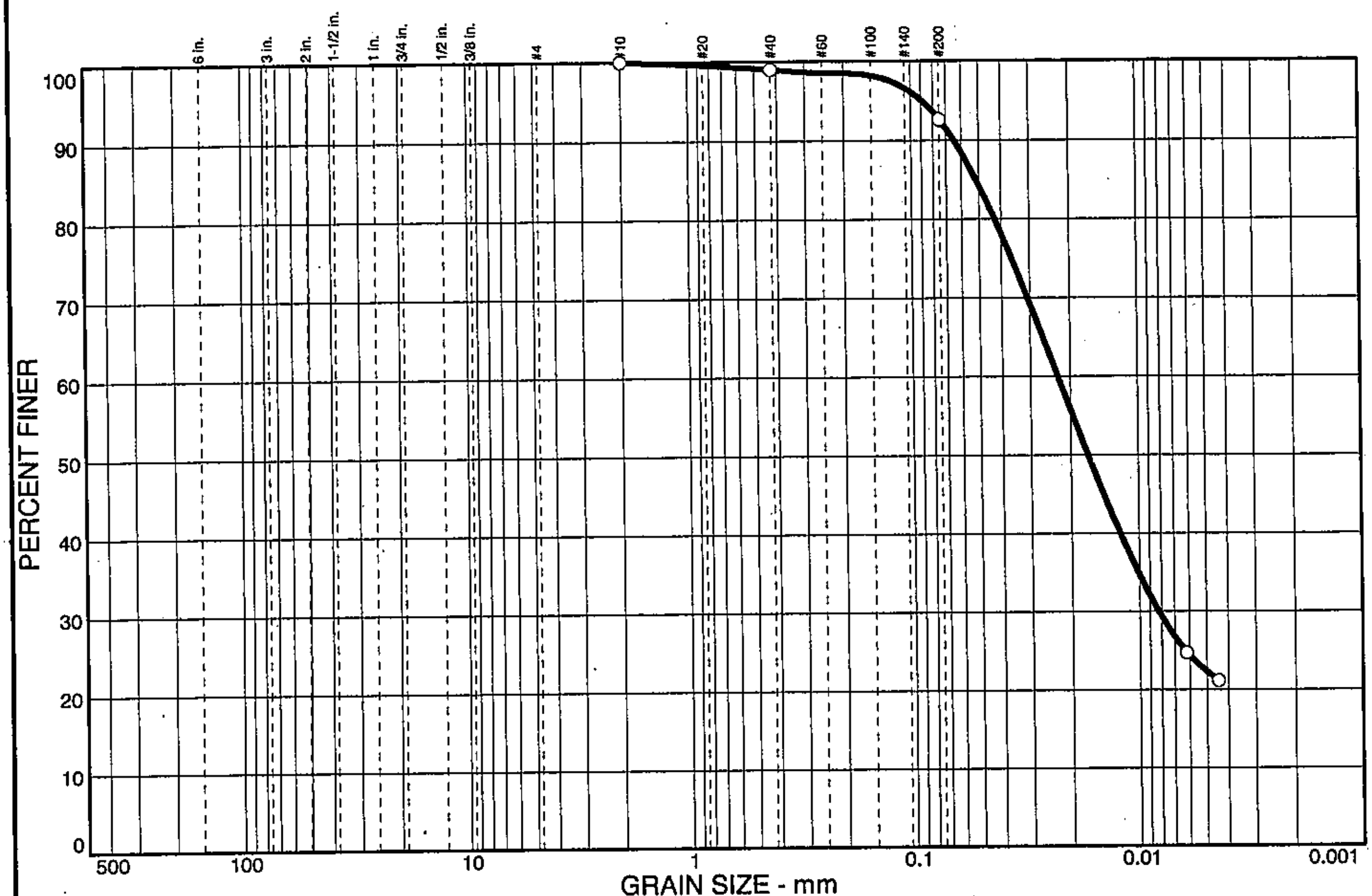
Moisture Content= 24.0%

Sample No.: 3 **Source of Sample:** TR-44 **Date:** 3/29/05
Location: **Elev./Depth:** 6.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	1.0	6.4	70.3	22.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.0		
#200	92.6		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 0.0519 D_{60}= 0.0228 D_{50}= 0.0168
D_{30}= 0.0083 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(7)

Remarks

Moisture Content= 11.2%

Sample No.: 4 **Source of Sample:** TR-44 **Date:** 3/29/05
Location: **Elev./Depth:** 8.5

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00

Project No: 0121-3070.03 **Figure**

APPENDIX IV

MSE Wall Bearing Capacity and Stability Calculations
MSE Wall Settlement Calculations – Forward Abutment
MSE Wall Global Stability Results
Drilled Shaft – End Bearing and Side Resistance Calculations

APPENDIX IV
Rear Abutment – Native Soil Foundation

MSE Wall Bearing Capacity and Stability Calculations

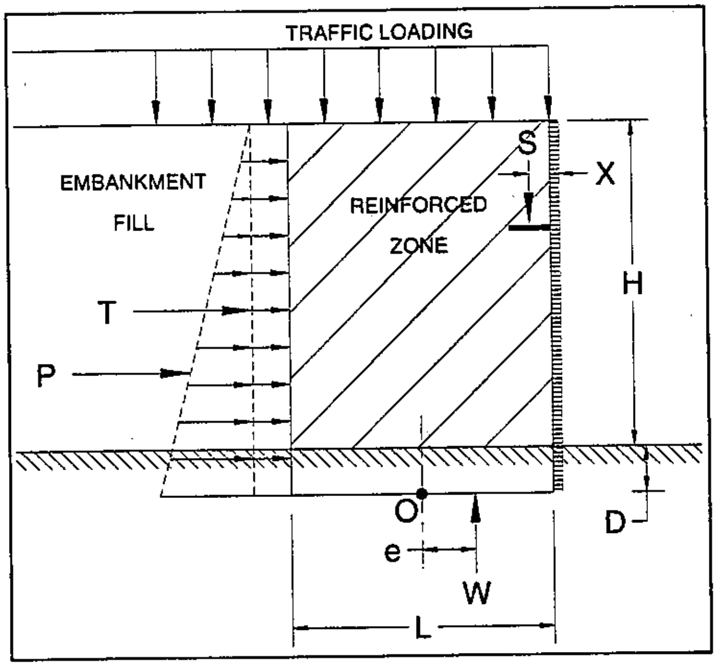
DLZ

SUBJECT

Client	TranSystems	JOB NUMBER	0121-3070.03
Project	SCI 823-0.00	SHEET NO.	1 OF 26
Item	Bearing Capacity (Rear Abutment)	COMP. BY	WMA DATE 1/10/07
SCI-823 over SR 140 (Webster St)		CHECKED BY	SJR DATE 1-11-07
Spread Footing founded in MSE fill		Native Soil Foundation	

BEARING CAPACITY OF A MSE WALL (Bridge Supported on Spread Footings)

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2250	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	39.5	ft	Height of wall
H+D	=	43	ft	
L factor	=	0.95		Length factor-range (0.7 - 1.0)
L=B	=	41	ft	Length of MSE reinforcement
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	21.5	ft	Γ Pa	=	14.3	ft
				Γ S	=	13.0	ft

B'	=	32.72	ft	
γ'	=	57.6	pcf	
W_t		9,840	lb/ft of wall	Weight from traffic
W_{mse}	=	211,560	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE} + S}{L - 2e}$$

σ_v = 7,867 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 11,738 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 4,695 psf

Factor of Safety = **1.49** No Good

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 21,066 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 8,426 psf

Factor of Safety = **2.68** OK

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 4.14 ft

Kern

e < L/6 = 6.83 ft

DLZ

SUBJECT	Client TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
	Project SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 26
	Item MSE Wall Stability-Rear Abutment	COMP. BY WMA	DATE 01/10/07
	SCI-823 over SR 140 (Webster St)	CHECKED BY SAK	DATE 1-11-07
	Spread Footing founded in MSE fill	Native Soil Foundation	

STABILITY OF MSE WALL

Assumptions:

1. Estimated height of embankment; H=43'
2. It is assumed that the bridge is supported on spread footings
3. Ground water; Dw=0.0'
4. Traffic loading is neglacted in resisting forces
5. Neglect Structure Loading in resisting forces

Wall Properties

H+D	=	43	feet
γ_{mse}	=	120	pcf
L	=	41	feet
L factor	=	0.95	
ϕ	=	30	deg

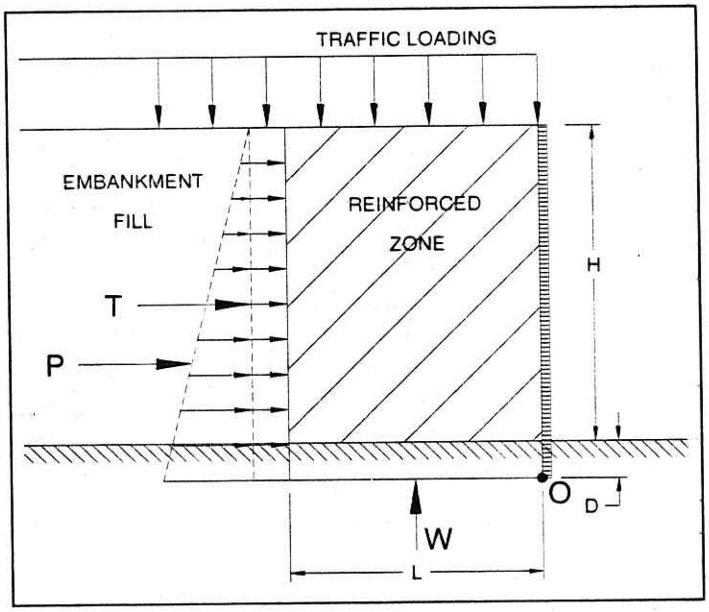
Foundational Soil Properties

c	=	2250	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where: $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 40,016 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 74,046 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 92,250 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$

Calculated: $\boxed{FS = 1.85}$ Required: FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 4,336,980 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 597,967 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

Calculated: $\boxed{FS = 7.25}$ Required: FS = 2.00

Resistance Against Overturning is **OK**

DLZ

SUBJECT

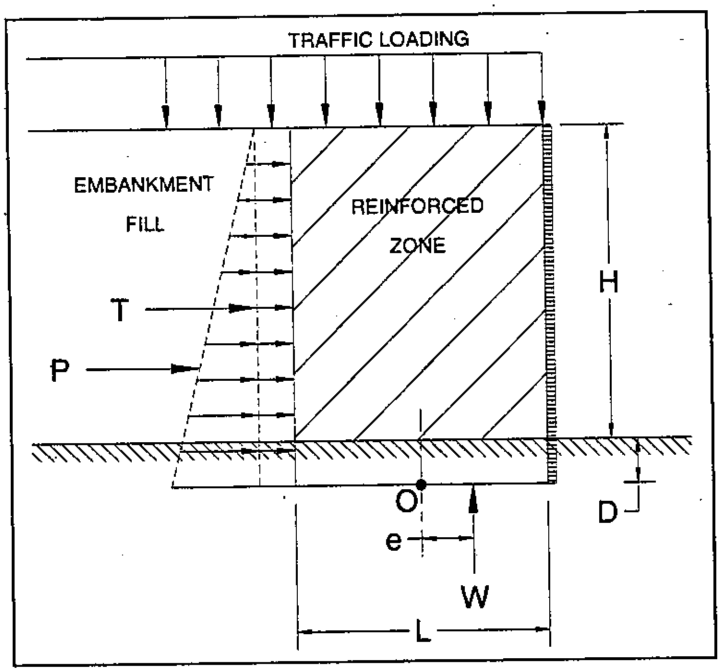
Client	TranSystems	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	3 OF 26
Item	Bearing Capacity-Rear Abutment	COMP. BY	WMA DATE 12/28/06
SCI-823 over SR 140 (Webster St)		CHECKED BY	SAR DATE 1-5-07

Assumes Piles

Native Soil Foundation

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2250	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	39.5	ft	Height of wall
H+D	=	43	ft	
L factor	=	0.8		Length factor-range (0.7 - 1.0)
L=B	=	35	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	14.333	ft	Moment arm
Γ Wt	=	21.5	ft	Moment arm
B'	=	28.68	ft	
γ'	=	57.6	pcf	
W_t		8,400	lb/ft of wall	Weight from traffic
W_{mse}	=	180,600	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L-2e}$$

σ_v = 6,590 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 11,738 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 4,695 psf

Factor of Safety = **1.78** No Good

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 18,815 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 7,526 psf

Factor of Safety = **2.86** OK

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 3.16 ft

Kern

e < L/6 = 5.83 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	4 OF 26
Item	MSE Wall Stability-Rear Abutment	COMP. BY	WMA DATE 12/28/06
	SCI-823 over SR 140 (Webster St)	CHECKED BY	SJR DATE 1-5-07

Assumes Piles

Native Soil Foundation

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=43'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

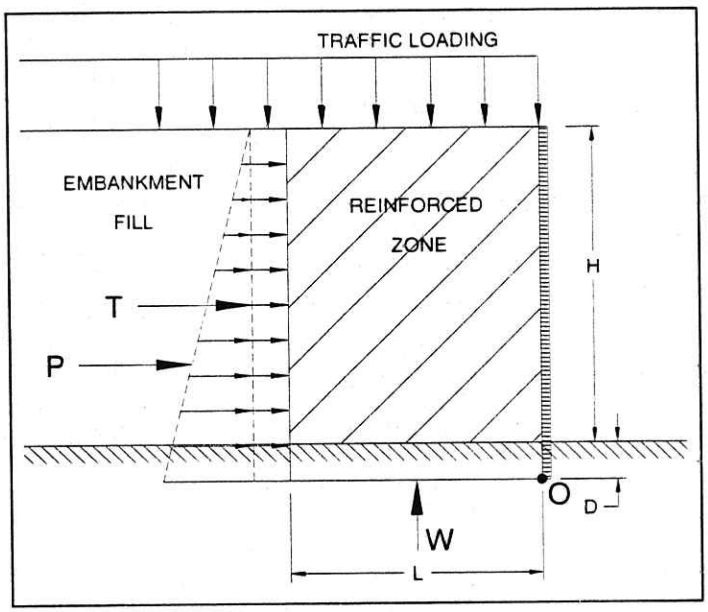
Wall Properties

$H+D$ = 43 feet
γ_{mse} = 120 pcf
L = 35 feet
L factor = 0.80
ϕ = 30 deg

Foundational Soil Properties

c = 0 psf Cohesion
ϕ' = 29 deg Friction angle
ω_T = 240 psf Traffic loading
Length factor-range (0.7 - 1.0)
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE



Thrust: $$P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 40,016 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 63,210 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$$FS = \frac{P_r}{P_a}$$

Calculated: **FS = 1.58**

Required: FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 3,160,500 lb-ft

$$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 597,967 lb-ft

$$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

Calculated: **FS = 5.29**

Required: FS = 2.00

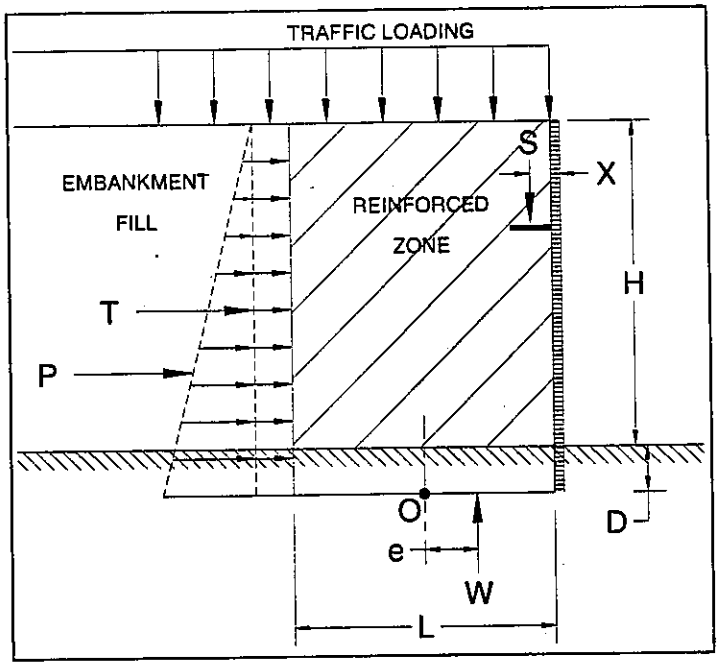
Resistance Against Overturning is **OK**

APPENDIX IV
Rear Abutment – Granular Fill Foundation

MSE Wall Bearing Capacity and Stability Calculations

BEARING CAPACITY OF A MSE WALL (Bridge Supported on Spread Footings)

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	39.5	ft	Height of wall
H+D	=	43	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	31	ft	Length of MSE reinforcement
Ka	=	0.33		

Force Moment Arms

				Γ Pa	=	14.3	ft
Γ Wt	=	21.5	ft	Γ S	=	8.0	ft

B'	=	22.30	ft	
γ'	=	57.6	pcf	
W_t		7,440	lb/ft of wall	Weight from traffic
W_{mse}	=	159,960	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE} + S}{L - 2e}$$

σ_v = 9,121 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 31,458 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 12,583 psf

Factor of Safety = **3.45** OK

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 31,458 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 12,583 psf

Factor of Safety = **3.45** OK

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 4.35 ft

Kern

e < L/6 = 5.17 ft

DLZ

SUBJECT

Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	6 OF 26
Item	MSE Wall Stability-Rear Abutment	COMP. BY	WMA DATE 01/10/07
SCI-823 over SR 140 (Webster St)		CHECKED BY	SJK DATE 1-11-07

Spread Footing Founded in MSE fill **Granular Fill Soil Foundation**

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=43'
2 It is assumed that the bridge is supported on spread footings
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5 Neglect Structure Loading in resisting forces

Wall Properties

H+D	=	43	feet
γ_{mse}	=	120	pcf
L	=	31	feet
L factor	=	0.70	
ϕ	=	30	deg

Foundational Soil Properties

c	=	0	psf	Cohesion
ϕ'	=	34	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45-\frac{\phi}{2})$ K_a = 0.33

P_a = 40,016 lbs per foot of wall

Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 {AASHTO, Bridge Design Manual, 303.4.1.1}

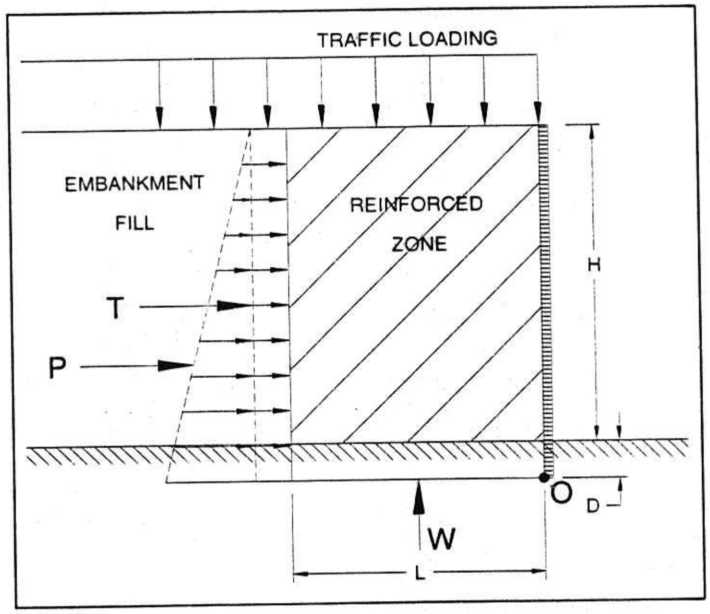
P_r = 71,982 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value



$FS = \frac{P_r}{P_a}$

Calculated: FS = **1.80**

Required: FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,479,380 lb-ft

$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 597,967 lb-ft

$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$

Calculated: FS = **4.15**

Required: FS = 2.00

Resistance Against Overturning is **OK**

DLZ

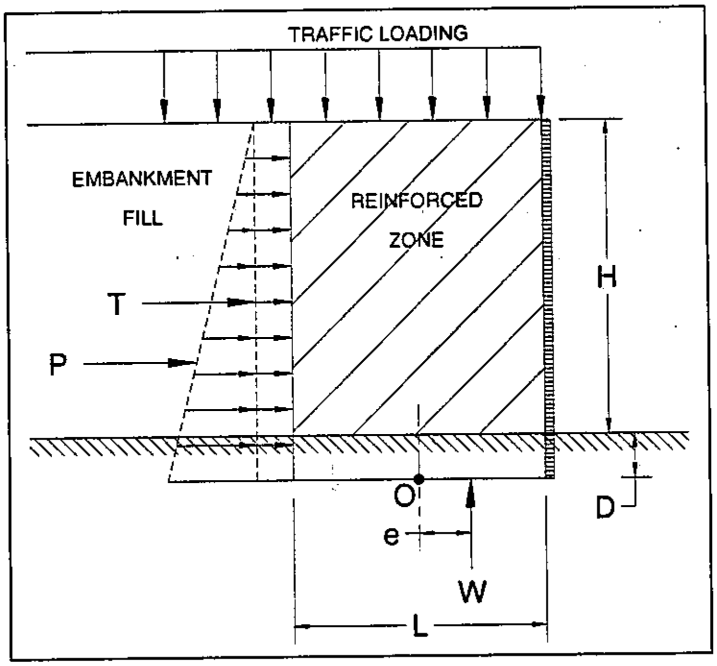
SUBJECT			JOB NUMBER	0121-3070.03		
	Client	TranSystems	SHEET NO.	7	OF	26
	Project	SCI-823 Portsmouth Bypass	COMP. BY	WMA	DATE	12/28/06
	Item	Bearing Capacity-Rear Abutment	CHECKED BY	SJK	DATE	1-5-07
	SCI-823 over SR 140 (Webster St)					

Assumes Piles

Granular Fill Soil Foundation

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	39.5	ft	Height of wall
H+D	=	43	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	31	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	14.333	ft	Moment arm
Γ Wt	=	21.5	ft	Moment arm
B'	=	23.86	ft	
γ'	=	57.6	pcf	
W_t		7,440	lb/ft of wall	Weight from traffic
W_{mse}	=	159,960	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 7,016 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 33,302 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 13,321 psf

Factor of Safety = 4.75 **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 33,302 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 13,321 psf

Factor of Safety = 4.75 **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 3.57 ft

Kern

e < L/6 = 5.17 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	8 OF 26
Item	MSE Wall Stability-Rear Abutment	COMP. BY	WMA DATE 12/28/06
SCI-823 over SR 140 (Webster St)		CHECKED BY	SJK DATE 1-5-07

Assumes Piles

Granular Fill Soil Foundation

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=43'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

Wall Properties

H+D	=	43	feet
γ_{mse}	=	120	pcf
L	=	31	feet
L factor	=	0.70	
ϕ	=	30	deg

Foundational Soil Properties

c	=	0	psf	Cohesion
ϕ'	=	34	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

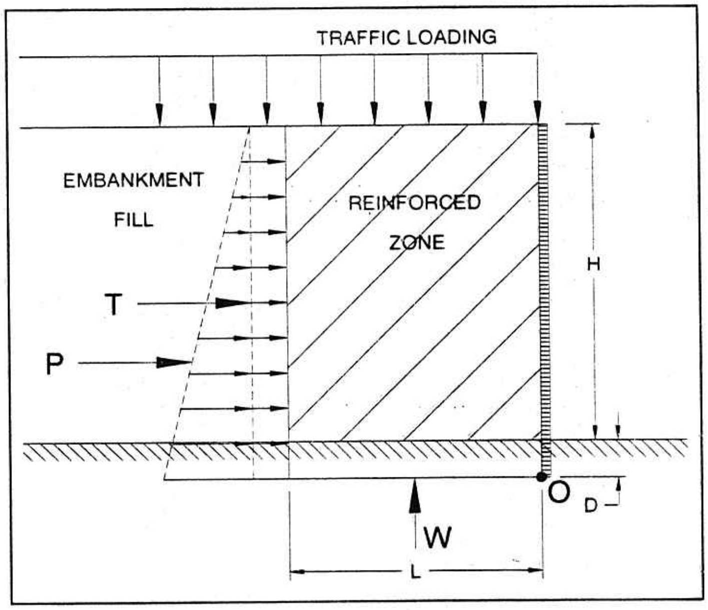
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 40,016 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 71,982 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$ Calculated $\boxed{FS = 1.80}$ Required FS = 1.50 Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,479,380 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 597,967 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated $\boxed{FS = 4.15}$ Required FS = 2.00 Resistance Against Overturning is **OK**

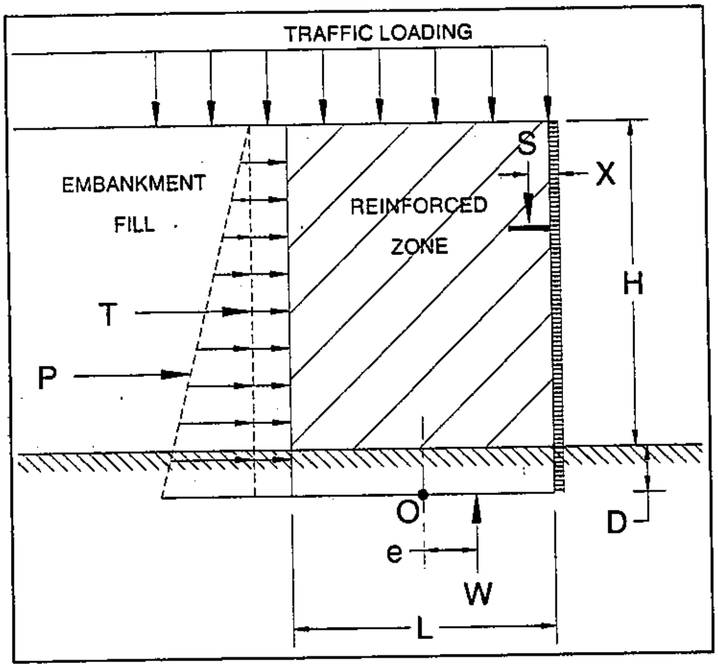
APPENDIX IV
Forward Abutment – Native Soil Foundation

MSE Wall Bearing Capacity and Stability Calculations

DLZ SUBJECT	Client TranSystems	JOB NUMBER	0121-3070.03
	Project SCI 823-0.00	SHEET NO.	9 OF 26
	Item Bearing Capacity (Forward Abutments)	COMP. BY	WMA DATE 1/10/07
	SCI-823 over SR 140 (Webster St)	CHECKED BY	5JK DATE 1-11-07
	Spread Footing founded in MSE fill	Native Soil Foundation	

BEARING CAPACITY OF A MSE WALL (Bridge Supported on Spread Footings)

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1750	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	32	ft	Height of wall
H+D	=	35	ft	
L factor	=	0.95		Length factor-range (0.7 - 1.0)
L=B	=	34	ft	Length of MSE reinforcement
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	17.5	ft	Γ Pa	=	11.7	ft
				Γ S	=	9.5	ft

B'	=	26.78	ft	
γ'	=	57.6	pcf	
W_t		8,160	lb/ft of wall	Weight from traffic
W_{mse}	=	142,800	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE} + S}{L - 2e}$$

σ_v = 6,981 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 9,168 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 3,667 psf

Factor of Safety = **1.31** $\boxed{\text{No Good}}$

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 17,757 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 7,103 psf

Factor of Safety = **2.54** $\boxed{\text{OK}}$

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 3.61 ft

Kern

e < L/6 = 5.67 ft

DLZ

SUBJECT		JOB NUMBER	0121-3070.03
Client	TranSystems ODOT D-9	SHEET NO.	10 OF 26
Project	SCI-823 Portsmouth Bypass	COMP. BY	WMA DATE 01/10/07
Item	MSE Wall Stability-Forward Abutment	CHECKED BY	SJK DATE 1-11-07
	SCI-823 over SR 140 (Webster St)		

Spread Footing founded in MSE fill **Native Soil Foundation**

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=35'
2 It is assumed that the bridge is supported on spread footings
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5 Neglect Structure Loading in resisting forces

Wall Properties

H+D	=	35	feet
γ_{mse}	=	120	pcf
L	=	34	feet
L factor	=	0.95	
ϕ	=	30	deg

Foundational Soil Properties

c	=	1750	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

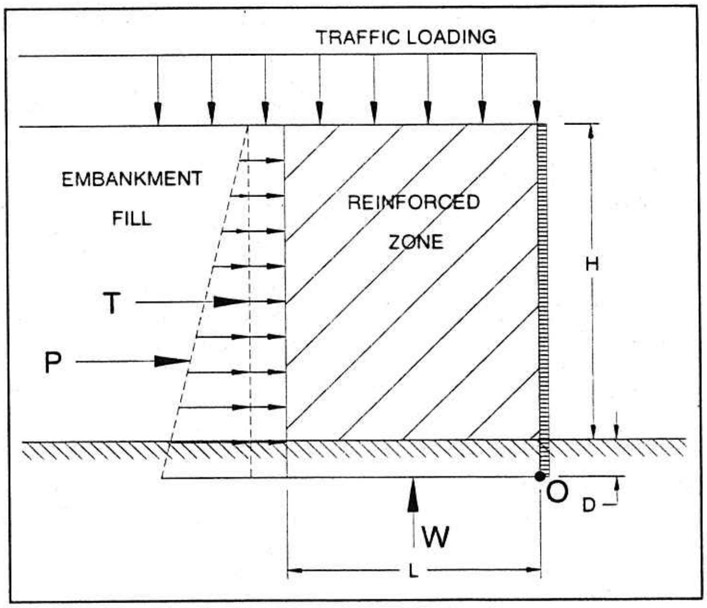
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 27,027 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 49,980 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 59,500 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$ Calculated: **FS = 1.85** Required: FS = 1.50 Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,427,600 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

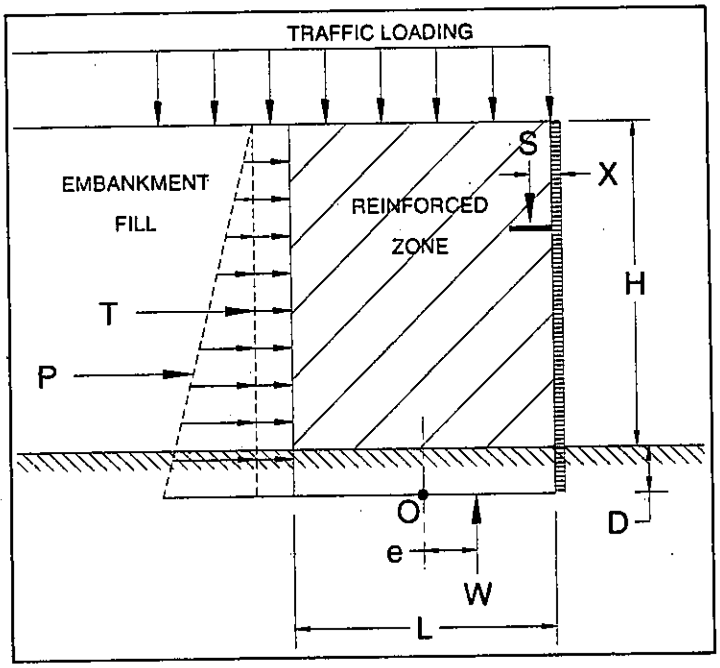
$\Sigma M_{overturning}$ = 331,485 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated: **FS = 7.32** Required: FS = 2.00 Resistance Against Overturning is **OK**

DLZ	SUBJECT			
	Client TranSystems		JOB NUMBER	0121-3070.03
	Project SCI 823-0.00		SHEET NO.	11 OF 26
	Item Bearing Capacity (Forward Abutments)		COMP. BY	WMA DATE 1/10/07
	SCI-823 over SR 140 (Webster St) Staged Construction		CHECKED BY	SJR DATE 1-11-07
	Spread Footing founded in MSE fill		Native Soil Foundation	

BEARING CAPACITY OF A MSE WALL (Bridge Supported on Spread Footings)

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1750	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	23	ft	**Max. Staged Height**
H+D	=	26	ft	
L factor	=	0.95		Length factor-range (0.7 - 1.0)
L=B	=	34	ft	Length of MSE reinforcement
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	13.0	ft	Γ Pa	=	8.7	ft
				Γ S	=	9.5	ft

B'	=	31.50	ft	
γ'	=	57.6	pcf	
W_t		8,160	lb/ft of wall	Weight from traffic
W_{mse}	=	106,080	lb/ft of wall	Weight from MSE wall
S	=	0	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE} + S}{L - 2e}$$

σ_v = 3,627 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = c N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 9,168 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 3,667 psf

Factor of Safety = 2.53 **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c' N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 20,386 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 8,154 psf

Factor of Safety = 5.62 **OK**

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.25 ft

Kern

e < L/6 = 5.67 ft

DLZ

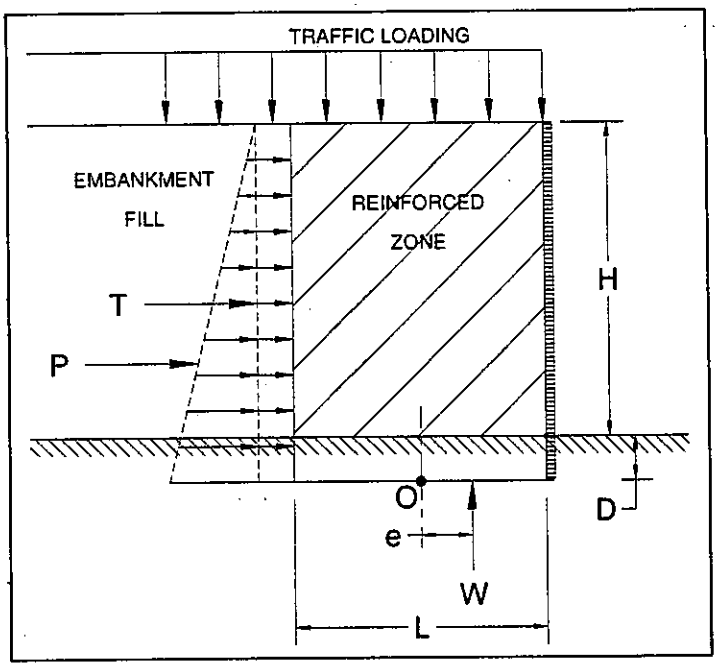
SUBJECT			
Client: TranSystems		JOB NUMBER	0121-3070.03
Project: SCI-823 Portsmouth Bypass		SHEET NO.	12 OF 76
Item: Bearing Capacity-Forward Abutment		COMP. BY	WMA DATE 12/28/06
SCI-823 over SR 140 (Webster St)		CHECKED BY	SJR DATE 1-5-07

Assumes Piles

Native Soil Foundation

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1750	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	32	ft	Height of wall
H+D	=	35	ft	
L factor	=	0.8		Length factor-range (0.7 - 1.0)
L=B	=	28	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	11.667	ft	Moment arm
Γ Wt	=	17.5	ft	Moment arm
B'	=	22.66	ft	
γ'	=	57.6	pcf	
W_t		6,720	lb/ft of wall	Weight from traffic
W_{mse}	=	117,600	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 5,486 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 9,168 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 3,667 psf

Factor of Safety = 1.67 | **No Good**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 15,462 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 6,185 psf

Factor of Safety = 2.82 | **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 2.67 ft

Kern

e < L/6 = 4.67 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	13 OF 26
Item	MSE Wall Stability-Forward Abutment	COMP. BY	WMA DATE 12/28/06
SCI-823 over SR 140 (Webster St)		CHECKED BY	SAR DATE 1-5-07

Assumes Piles

Native Soil Foundation

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=35'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

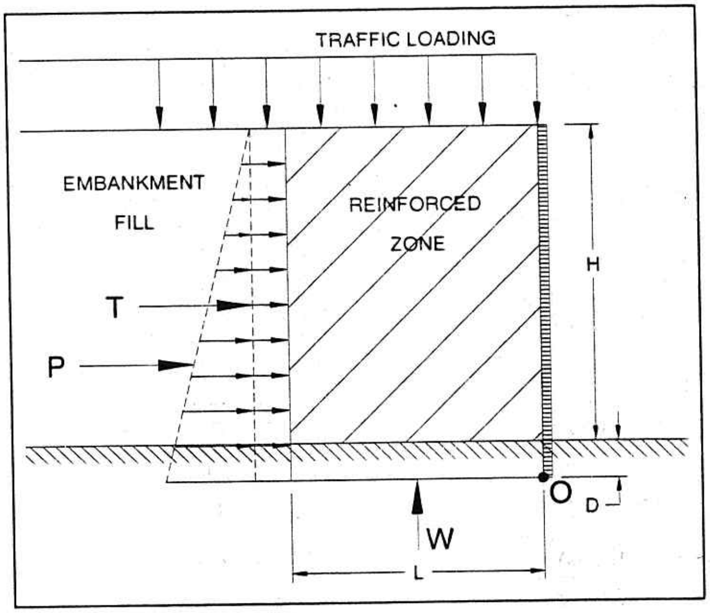
Wall Properties			Foundational Soil Properties			
H+D =	35	feet	c =	0	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	29	deg	Friction angle
L =	28	feet	ω_T =	240	psf	Traffic loading
L factor =	0.80		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where: $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 27,027 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 41,160 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$

Calculated: FS = **1.52**

Required: FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 1,646,400 lb-ft

$$\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 331,485 lb-ft

$$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

Calculated: FS = **4.97**

Required: FS = 2.00

Resistance Against Overturning is **OK**

SUBJECT	Client TranSystems	JOB NUMBER	0121-3070.03
	Project SCI-823 Portsmouth Bypass	SHEET NO.	14 OF 26
	Item Bearing Capacity-Forward Abutment	COMP. BY	WMA DATE 12/28/06
	SCI-823 over SR 140 (Webster St)	CHECKED BY	SJR DATE 1-5-07

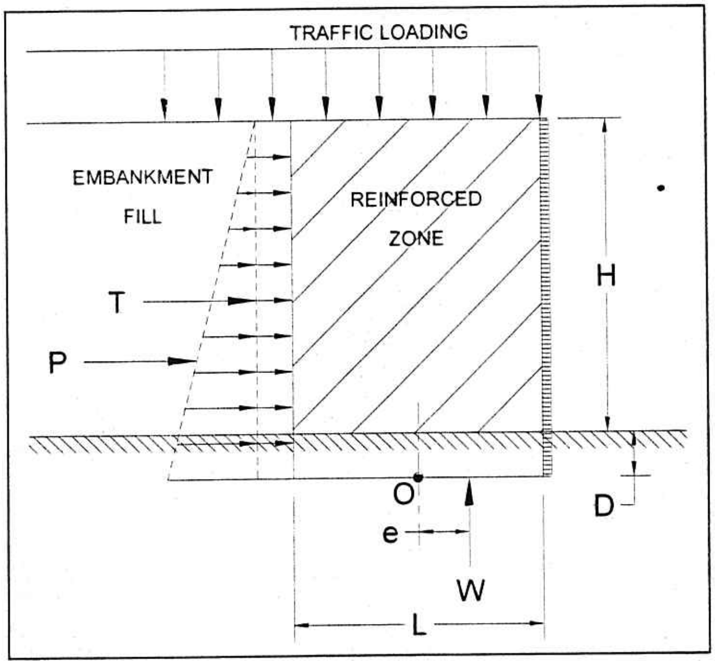
DLZ

Assumes Piles

Native Soil Foundation Staged

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1750	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	22	ft ✓	**Max. Staged Height**
H+D	=	25	ft	
L factor	=	0.8		Length factor-range (0.7 - 1.0)
L=B	=	28	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	8.3333	ft	Moment arm
Γ Wt	=	12.5	ft	Moment arm
B'	=	25.18	ft	
γ'	=	57.6	pcf	
W_t		6,720	lb/ft of wall	Weight from traffic
W_{mse}	=	84,000	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 3,603 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 9,168 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 3,667 psf

Factor of Safety = **2.54** **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 16,866 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 6,746 psf

Factor of Safety = **4.68** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.41 ft

Kern

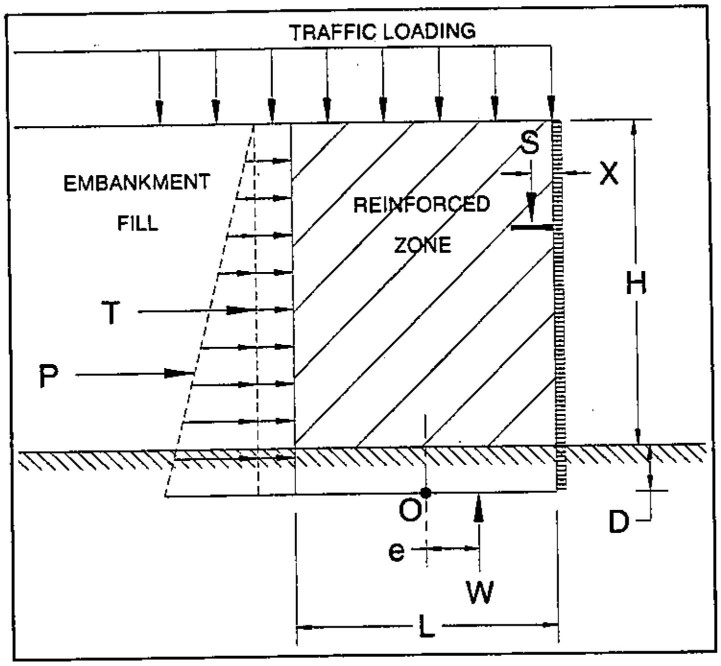
e < L/6 = 4.67 ft

APPENDIX IV
Forward Abutment – Granular Fill Foundation

MSE Wall Bearing Capacity and Stability Calculations

BEARING CAPACITY OF A MSE WALL (Bridge Supported on Spread Footings)

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE} + S}{L - 2e}$$

σ_v = 8,149 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 26,420 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 10,568 psf

Factor of Safety = **3.24** OK

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 26,420 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 10,568 psf

Factor of Safety = **3.24** OK

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	32	ft	Height of wall
H+D	=	35	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	25	ft	Length of MSE reinforcement
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	17.5	ft	Γ Pa	=	11.7	ft
				Γ S	=	5.0	ft

B'	=	18.04	ft	
γ'	=	57.6	pcf	
W_t		6,000	lb/ft of wall	Weight from traffic
W_{mse}	=	105,000	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 3.48 ft

Kern

e < L/6 = 4.17 ft

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portmouth Bypass	SHEET NO.	16 OF 26
Item	MSE Wall Stability-Forward Abutment	COMP. BY	WMA DATE 01/10/07
	SCI-823 over SR 140 (Webster St)	CHECKED BY	SJK DATE 1-11-07

Spread Footing founded in MSE fill — **Granular Fill Foundation**

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=35'
2 It is assumed that the bridge is supported on spread footings
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5 Neglect Structure Loading in resisting forces

Wall Properties

H+D	=	35	feet
γ_{mse}	=	120	pcf
L	=	25	feet
L factor	=	0.70	
ϕ	=	30	deg

Foundational Soil Properties

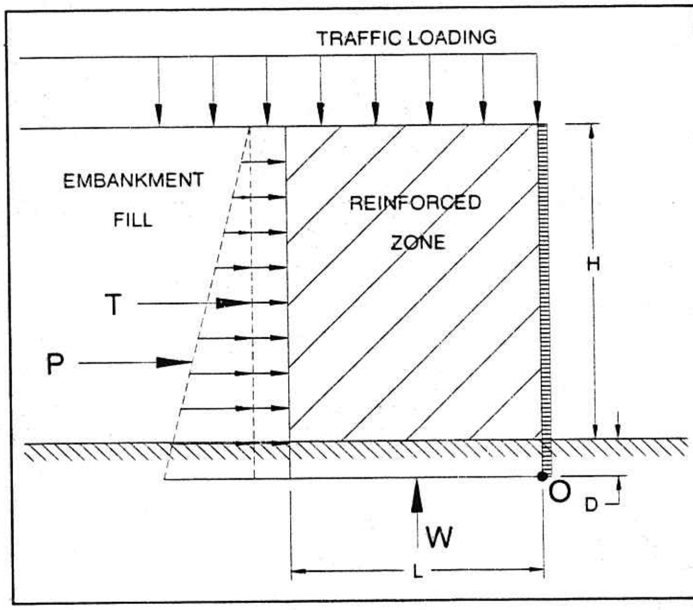
c	=	0	psf	Cohesion
ϕ'	=	34	deg	Friction angle
ω_T	=	240	psf	Traffic loading
Length factor-range (0.7 - 1.0)				
Friction Angle of Embankment Fill				

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where: $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 27,027 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where: $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 47,250 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$ Calculated: **FS = 1.75** Required: FS = 1.50 Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 1,312,500 lb-ft $\qquad \Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 331,485 lb-ft $\qquad \Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated: **FS = 3.96** Required: FS = 2.00 Resistance Against Overturning is **OK**

DLZ

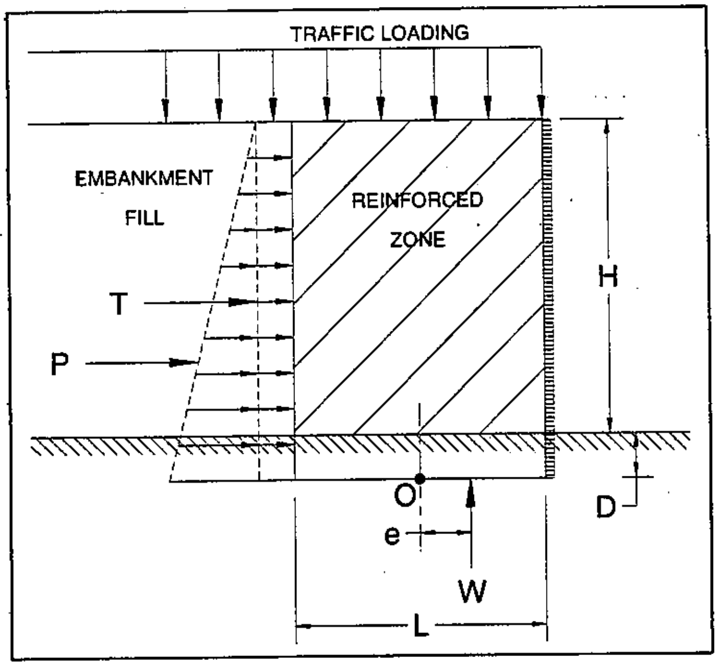
SUBJECT			
Client	TranSystems	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	17 OF 26
Item	Bearing Capacity-Forward Abutment	COMP. BY	WMA DATE 12/28/06
	SCI-823 over SR 140 (Webster St)	CHECKED BY	SAR DATE 1-5-07

Assumes Piles

Granular Fill Soil Foundation

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3	ft	Embedment depth (H/20<D<3.0)
Dw	=	0	ft	Groundwater depth
H	=	32	ft	Height of wall
H+D	=	35	ft	
L factor	=	0.7		Length factor-range (0.7 - 1.0)
L=B	=	25	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	11.667	ft	Moment arm
Γ Wt	=	17.5	ft	Moment arm
B'	=	19.02	ft	
γ'	=	57.6	pcf	
W_t		6,000	lb/ft of wall	Weight from traffic
W_{mse}	=	105,000	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 5,836 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 27,579 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 11,032 psf

Factor of Safety = **4.73** OK

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma B N_\gamma$$

q_{ULT} = 27,579 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 11,032 psf

Factor of Safety = **4.73** OK

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 2.99 ft

Kern

e < L/6 = 4.17 ft

DLZ	SUBJECT	Client: TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
		Project: SCI-823 Portmouth Bypass	SHEET NO.	18 OF 26
		Item: MSE Wall Stability-Forward Abutment	COMP. BY	WMA DATE 12/28/06
		SCI-823 over SR 140 (Webster St)	CHECKED BY	SJR DATE 1-5-07

Assumes Piles

Granular Fill Soil Foundation

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=35'
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

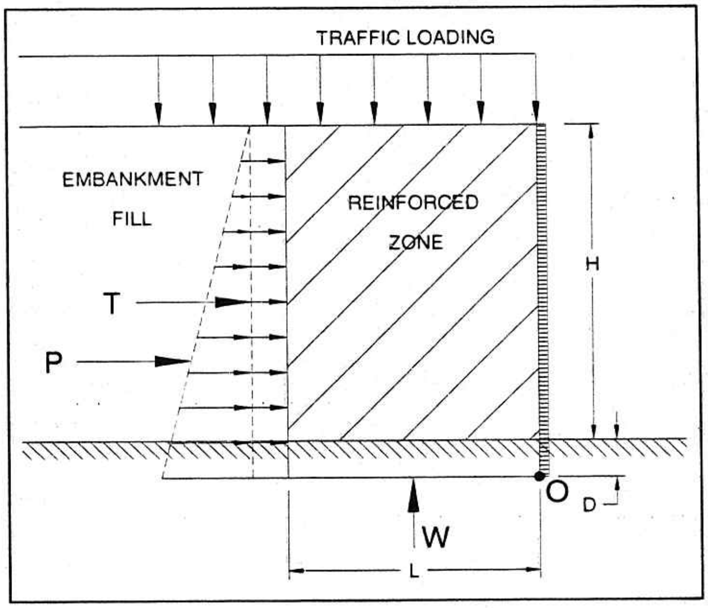
Wall Properties			Foundational Soil Properties			
H+D =	35	feet	c =	0	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	34	deg	Friction angle
L =	25	feet	ω_T =	240	psf	Traffic loading
L factor =	0.70		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where: $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 27,027 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.45

0.67μ Max. = 0.55 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 47,250 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$

Calculated	Required	
FS = **1.75**	FS = 1.50	Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 1,312,500 lb-ft $\qquad \Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 331,485 lb-ft $\qquad \Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

Calculated	Required	
FS = **3.96**	FS = 2.00	Resistance Against Overturning is **OK**

APPENDIX IV

MSE Wall Settlement Calculations – Forward Abutment

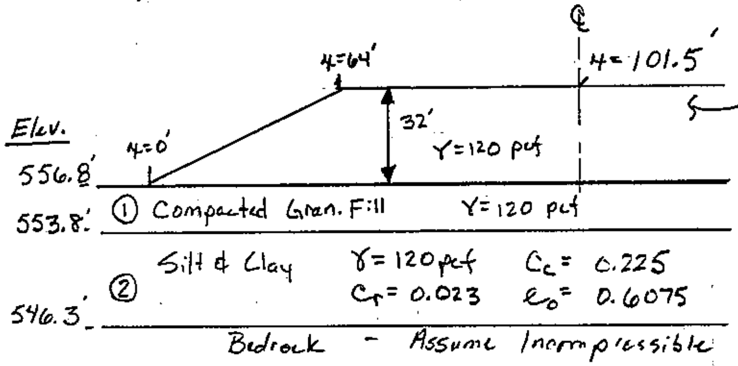
DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: TranSystems / ODOT D-9
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Consolidation Parameters
SR 823 over SR 140 Forward Abutment

PROJECT NO. 0121-3070.03

COMP. BY SJR DATE 1-4-07
CHECKED BY SWT DATE 1-5-07

Most Critical soil profile taken from boring B-16
At forward abutment, maximum embankment height is approximately 32'



} Assume Incompressible
} Consolidation Parameters estimated from FHWA NHI-00-045

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*From EMBANK - Using "End of Fill" condition to model MSE wall.

at Centerline ($x = 101.5'$) $\delta_c \simeq 0.83''$

at TOE of wall ($x = 0'$) $\delta \simeq 0.06''$

Differential Settlement : $DS = \dfrac{0.83'' - 0.06''\,(1'/12'')}{101.5'} = 6.3\times10^{-4} < 1.0\%$ ✓ $\boxed{OK}$

Sample Calculations for estimating consolidation parameters
From FHWA NHI-00-045 Average $\omega = 22.5\%$

1.) Check if overconsolidated [B-16 Clay layer] $\dfrac{\omega - PL}{LL - PL} = \dfrac{25-26}{44-26} = -0.06 < 0.7$

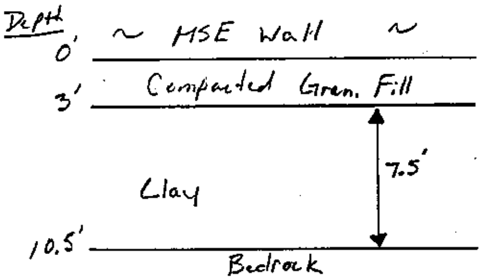
May assume Preconsolidated

2.) Initial Void Ratio $e_o = \dfrac{G_s \cdot \omega}{100} = \dfrac{(2.70)(22.5)}{100} = 0.6075$

3.) Compression Index $C_c = \dfrac{\omega}{100} = \dfrac{22.5}{100} = 0.225$

Recompression Index $C_r = \dfrac{\omega}{1000} = \dfrac{22.5}{1000} = 0.023$

Based on boring B-16 - Consider Clay Layer (more Conservative)



* Assume Single Drainage, H = 7.5'

LL = 44 → From FHWA HI-97-021 [NAVFAC, DM-7.1, 1982]
$C_v \simeq 0.2$ ft²/day

For $U = 90\%$ → $T = 0.848$

$$t_{90} = \frac{(0.848)(7.5')^2}{0.2 \text{ ft}^2/\text{day}} = \underline{238 \text{ days}}$$

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name	: SCI-823	Client	: TranSystems
File Name	: SR140	Project Manager	: Nix
Date	: 1/ 4/10	Computed by	: SJR

Settlement for X-Direction

Embank. slope, x direc.	= 64.00 (ft)	Height of fill H	= 32.00 (ft)
y direc.	= 64.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 75.00 (ft)	p load/unit area	= 3840.00 (psf)
Embankment bottom width	= 203.00 (ft)	Foundation Elev.	= 556.80 (ft)
Ground Surface Elev.	= 556.80 (ft)		
Water table Elev.	= 553.80 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	3.0	-----	-----	-----	120.00	----	----
2	COMP.	7.5	0.225	0.023	0.000	120.00	2.65	0.60

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	7.50	550.05	576.00	5100.00

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	20.30 Sett. (in.)	X = Stress (psf)	40.60 Sett. (in.)	X = Stress (psf)	60.90 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.	1231.89	0.64	1817.64	0.80
2	59.42	0.06	619.64	0.41				
		------		------		------		------
		0.06		0.41		0.64		0.80

Layer	X = Stress (psf)	81.20 Sett. (in.)	X = Stress (psf)	101.50 Sett. (in.)
1	INCOMP.	INCOMP.		
2	1935.08	0.83	1936.94	0.83
		------		------
		0.83		0.83

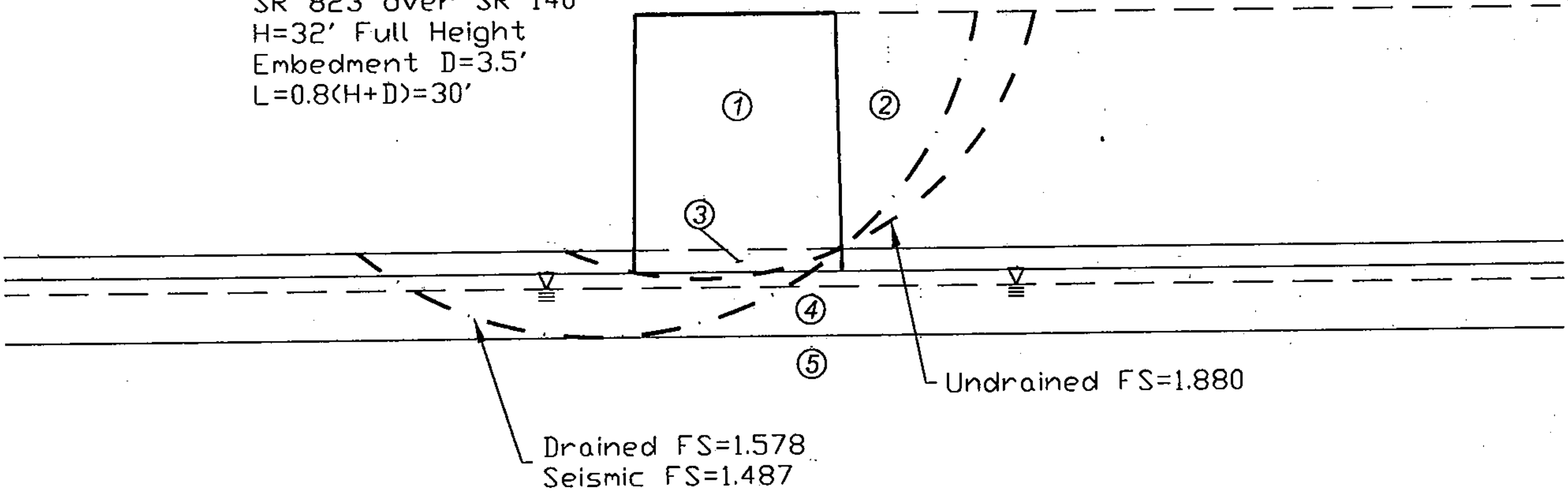
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

APPENDIX IV

MSE Wall Global Stability Results (Pile Supported Abutments)
MSE Wall Global Stability Results (Spread Footing Supported Abutments)

Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran./MSE Fill	0	34	0	34	120
Material 4	Stiff	Clay/Silty Clay	1750	0	0	29	120
Material 5		BEDROCK	5000	45	5000	45	145

MSE Stability Analysis
B-16 Profile
SR 823 over SR 140
H=32' Full Height
Embedment D=3.5'
L=0.8(H+D)=30'



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SR 823 over SR 140 (Webster St)
Forward Abutment Location
Based on Borings B-16, B-17, TR-73

MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 1/05/07

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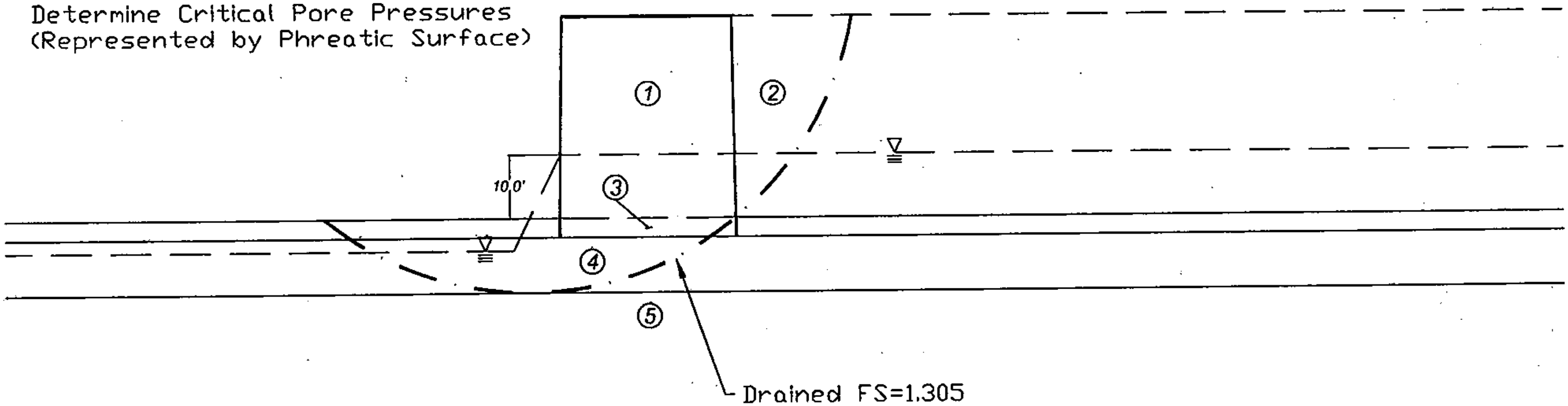
MSE Stability Analysis - Drained
B-16 Profile
SR 823 over SR 140
H=32' Full Height
Embedment D=3.0'
L=0.8(H+D)=28'

Material	Consistency	Soil Type	Undrained C (psf)	Undrained ϕ (deg)	Drained C' (psf)	Drained ϕ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran./MSE Fill	0	34	0	34	120
Material 4	Stiff	Clay/Silty Clay	1750	0	0	29	120
Material 5		BEDROCK	5000	45	5000	45	145

Determine Critical Pore Pressures
(Represented by Phreatic Surface)



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SR 823 over SR 140 (Webster St)
Forward Abutment Location
Based on Borings B-16, B-17, TR-73

CRITICAL PORE PRESSURE ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 1/05/07

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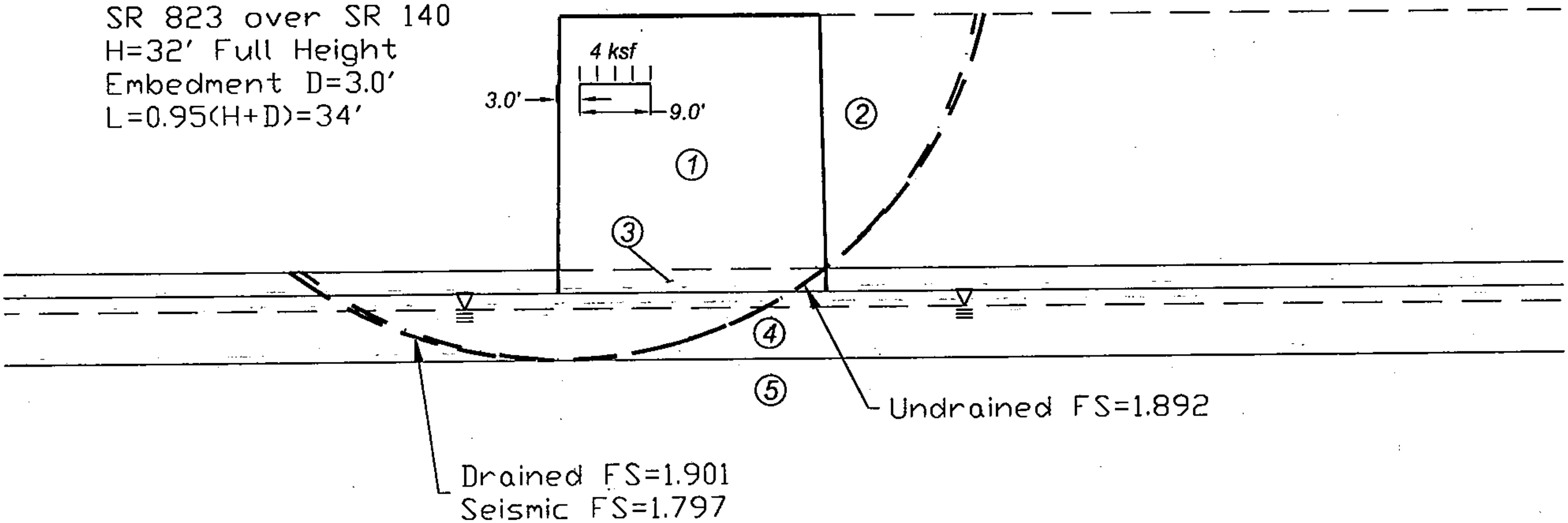
Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran./MSE Fill	0	34	0	34	120
Material 4	Stiff	Clay/Silty Clay	1750	0	0	29	120
Material 5		BEDROCK	5000	45	5000	45	145

MSE Stability Analysis
Using Spread Footings
to support Abutments
B-16 Profile
SR 823 over SR 140
H=32' Full Height
Embedment D=3.0'
L=0.95(H+D)=34'



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SR 823 over SR 140 (Webster St)
Forward Abutment Location
Based on Borings B-16, B-17, TR-73

MSE STABILITY ANALYSIS
Using Spread Footings
SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 1/11/07

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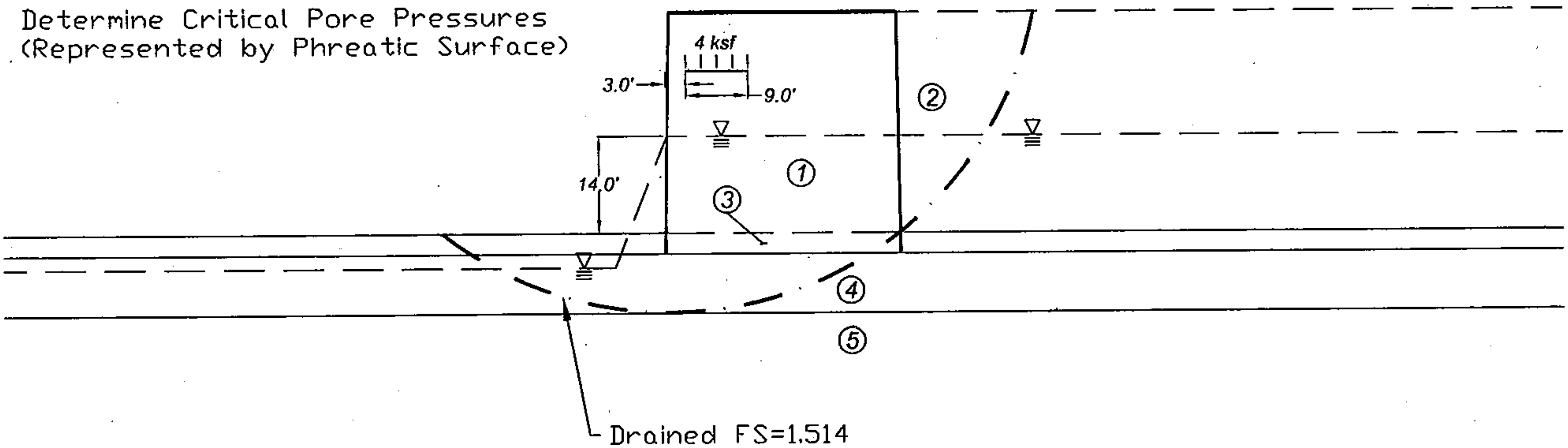
MSE Stability Analysis-Drained
Using Spread Footings
to support Abutments
B-16 Profile
SR 823 over SR 140
H=32' Full Height
Embedment D=3.0'
L=0.95(H+D)=34'

Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Compacted	Gran./MSE Fill	0	34	0	34	120
Material 4	Stiff	Clay/Silty Clay	1750	0	0	29	120
Material 5		BEDROCK	5000	45	5000	45	145

Determine Critical Pore Pressures
(Represented by Phreatic Surface)



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SR 823 over SR 140 (Webster St)
Forward Abutment Location
Based on Borings B-16, B-17, TR-73

CRITICAL PORE PRESSURE ANALYSIS
Using Spread Footings
SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 1/05/07

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APPENDIX IV

Drilled Shaft – End Bearing and Side Resistance Calculations

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: TranSystems Corp / ODOT D-9
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Drilled Shaft End Bearing & Friction SR 823 over SR 140 (Webster St.)

PROJECT NO. 0121-3070.02
SHEET NO. 26 OF 26
COMP. BY SJK DATE 1-5-07
CHECKED BY EWT DATE 1-5-07

* From lab testing rock core samples (lower bound) $q_u \approx 11{,}750$ psi

[FHWA - IF-99-025] Eq. 11.6 $q_{max}(MPa) = 4.83[q_u(MPa)]^{0.51}$

End Bearing * For RQD between 70-100 & $q_u > 5.2$ tsf

$$q_u = 11{,}750 \text{ psi} = 81.0 \text{ MPa}$$

[Eq. 11.6] $q_{max} = 4.83[q_u(MPa)]^{0.51}$

$$q_{max} = 4.83[81.0 \text{ MPa}]^{0.51} = 45.4 \text{ MPa} = 6{,}588 \text{ psi} = 948 \text{ ksf}$$

$$q_a = \frac{q_{max}}{FS} = \frac{948 \text{ ksf}}{3.0} = 316 \text{ ksf}$$

* For this type & quality Sandstone, we typically use;

$\boxed{\text{Use } q_{allow} = 80 \text{ ksf}}$ for Competent Rock

Side Friction

[FHWA - IF-99-025] Eq. 11.24 $f_{max} = 0.65\, p_a [q_u/p_a]^{0.5} \leq 0.65\, p_a [f'_c/p_a]^{0.5}$

*Assumes Smooth Rock Socket

$$f_{max} = 0.65(14.7 \text{ psi})\left[\frac{11{,}750 \text{ psi}}{14.7 \text{ psi}}\right]^{0.5} \leq 0.65(14.7 \text{ psi})\left[\frac{4500 \text{ psi}}{14.7 \text{ psi}}\right]^{0.5}$$

$f_{max} = 270.1$ psi ≤ 167.2 psi Use $f_{max} = 167$ psi

$$f_{allow} = \frac{167}{3.0} = 55.7 \text{ psi} = 8016 \text{ psf}$$

$\boxed{* \text{ Use } f_{allow} = 7{,}500 \text{ psf}}$ for Competent Rock

APPENDIX V

MSE Retaining Wall Design Parameters
Using Spread Footings

MSE Retaining Wall Parameters and Results of Analyses **Rear Abutment Location** **Native/Existing Soil Foundation – Spread Footing Foundations** ***Borings TR-44, TR-45, & B-15***
Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on F' = 30°)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37 Use (μ)(0.67) = 0.35 as a maximum value as per AASHTO, BDM, 303.4.1.1
Allowable Bearing Capacity – Undrained Condition **q_{all} = 4,695 psf**
Allowable Bearing Capacity – Drained Condition q_{all} = 8,426 psf
Global Stability (See Forward Abutment Analysis) Factor of Safety – Undrained Condition = >1.5 Factor of Safety – Drained Condition = >1.5 Factor of Safety – Seismic Condition = >1.3
Estimated Settlement of MSE volume Total settlement = 1 inch Differential settlement < 1/100 (See Forward Abutment Analysis)
Approximate Maximum Height of MSE Wall = 43.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = (H+D)(0.95) = 41 feet **Maximum Staged Construction Height = 22 feet** (See Forward Abutment Analysis) Maximum Pore Pressure* = 14.0 feet above ground surface

**Maximum pore pressure as measured in piezometer installed in clay layer. See results of analyses for more information. Assumes abutment is supported on spread footings.*

MSE Retaining Wall Parameters and Results of Analyses **Rear Abutment Location** **Granular Fill Foundation – Spread Footing Foundations** ***Borings TR-44, TR-45, & B-15***
Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on F' = 30°)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45 Use (μ)(0.67) = 0.55 as a maximum value as per AASHTO, BDM, 303.4.1.1
Allowable Bearing Capacity – Undrained Condition q_{all} = 12,583 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 12,583 psf
Global Stability (Founded on bedrock or on granular fill near bedrock) Factor of Safety – Undrained Condition >1.5 Factor of Safety – Drained Condition >1.5 Factor of Safety – Seismic Condition >1.3
Estimated Settlement of MSE volume Total settlement = Negligible (Undercut to bedrock, replace with compacted granular fill) Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 43.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = (H+D)(0.70) = 31 feet

MSE Retaining Wall Parameters and Results of Analyses **Forward Abutment Location** **Native/Existing Soil Foundation – Spread Footing Foundations** ***Borings TR-43, B-16, & B-17***
Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on F' = 30°)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37 Use (μ)(0.67) = 0.35 as a maximum value as per AASHTO, BDM, 303.4.1.1
Allowable Bearing Capacity – Undrained Condition **q_{all} = 3,667 psf**
Allowable Bearing Capacity – Drained Condition q_{all} = 7,103 psf
Global Stability Factor of Safety – Undrained Condition = 1.9 Factor of Safety – Drained Condition = 1.9 Factor of Safety – Seismic Condition = 1.8
Estimated Settlement of MSE volume Total settlement = 1 inch Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 35.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = (H+D)(0.95) = 34 feet **Maximum Staged Construction Height = 22 feet** Maximum Pore Pressure* = 14.0 feet above ground surface

**Maximum pore pressure as measured in piezometer installed in clay layer. See results of analyses for more information. Assumes abutment is supported on spread footings.*

MSE Retaining Wall Parameters and Results of Analyses **Forward Abutment Location** **Granular Fill Foundation – Spread Footing Foundations** ***Borings TR-43, B-16, & B-17***
Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on F' = 30°)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45 Use (μ)(0.67) = 0.55 as a maximum value as per AASHTO, BDM, 303.4.1.1
Allowable Bearing Capacity – Undrained Condition q_{all} = 10,568 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 10,568 psf
Global Stability (Founded on bedrock or on granular fill near bedrock) Factor of Safety – Undrained Condition >1.5 Factor of Safety – Drained Condition >1.5 Factor of Safety – Seismic Condition >1.3
Estimated Settlement of MSE volume Total settlement = Negligible (Undercut to bedrock, replace with compacted granular fill) Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 35.0 feet (including embedment) Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = (H+D)(0.70) = 25 feet